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SYNOPSIS The analysis of a tunnel section excavated through soft ground is presented. The tunnel construction was monitored and its behavior is compared with results obtained by a numerical simulation. The field instrumentation and the geotechnical properties of the site are presented and the modeling of the soil behavior using Lade's model discussed.

INTRODUCTION

In the last few years, São Paulo, the largest metropolitan region in Brazil and South America, has been extending its Metropolitan System, consisting in a set of surface and underground railways. At the present, a new subway line with 16.5 km is under construction and will connect to the existing North-South Line. One part of this line, named Paulista Line, was concluded in 1991. The Paulista Line is 4.5 km long and passes through an important commercial area. For this reason, this section was constructed underground by NATM and shield tunneling (Figure 1).

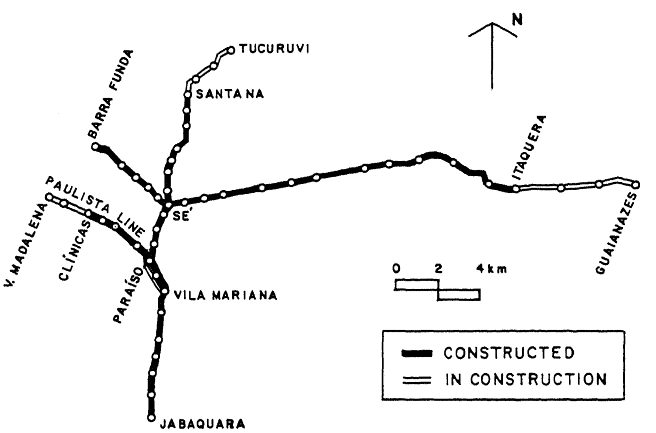

Fig. 1 The City of São Paulo subway system

One section of the Paulista Line, the Paraíso tunnel, is discussed in this paper. It is located near the Paraíso Station and was built using NATM. This tunnel, approximately 103 m in length, is characterized by a low depth of cover, ranging from a minimum of 6 m to a maximum of 9 m. At the instrumented section, the ground cover was about 7.6 m. The tunnel cross section is non-circular with a maximum height of 8.4 m and a maximum width of 11.4 m (82 m^2 net area), with primary and secondary shot-concrete support (20 cm and 15 cm thick), without invert (Figure 2).

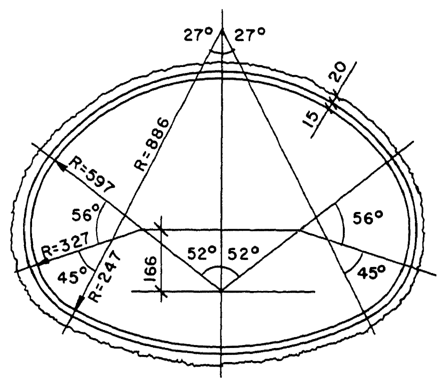

Fig. 2 Typical cross section of the Paraíso tunnel (dimensions in centimeters)

The construction of the Paraíso tunnel was monitored and this paper compares its performance with results obtained by a numerical simulation using the Finite Element Method. The field instrumentation is presented, as well as the description of the geology, the geotechnical properties of the site and the modeling of the soil behavior using Lade's model (Lade, 1977).

TUNNELING PROCEDURES AND FIELD INSTRUMENTATION

The tunnel construction was carried out by heading excavation and providing primary support consisting of I-shaped steel ribs (127 mm) and 20 cm shot-concrete (phase 1). The heading face was advanced leaving a central supporting ground core that was excavated after the heading support installation (phase 2). The face was excavated in 1.6 m segments. Similar procedures were continued in the tunneling of the bench (phase 3). The bench excavation were performed 7.0 m behind the heading face. After the tunnel completion, the shot-concret lining thickness was increased to 35 cm (Figure 3).

To evaluate the displacements, one slope indicator, 6 surface settlement points, 5 convergence pins and 2 deep settlement indicators were installed. Only the readings obtained from the slope indicator and from the surface settlement points were used for the purposes of this paper (Figure 3).

GEOLOGY AND GEOTECHNICAL PROPERTIES OF THE SITE

The subsoil of São Paulo City consists of quaternary and tertiary formations. At Paraíso tunnel, from ground level downwards there is a 2 m layer of fillings. Beneath that, there is a soft to overconsolidated stiff porous silty clay (Standard Penetration ranging from 4 to 11), about 9 m thick, called São Paulo porous red clay. The porous red clay sits on top of a heavily overconsolidated stiff and fissured sand clay (Standard Penetration ranging from 16 to 22), about 11 m thick, called variegated clay. These clays are tertiary and lateritic materials. Bellow the variegated clay there is a very stiff clay sand (Standard Penetration ranging from 28 to 35). The ground water table is in the frontier between the porous red clay and the variegated clay (12 m depth). The top half of the tunnel cross section was driven through the porous red clay and the rest through the variegated clay (Figure 3).

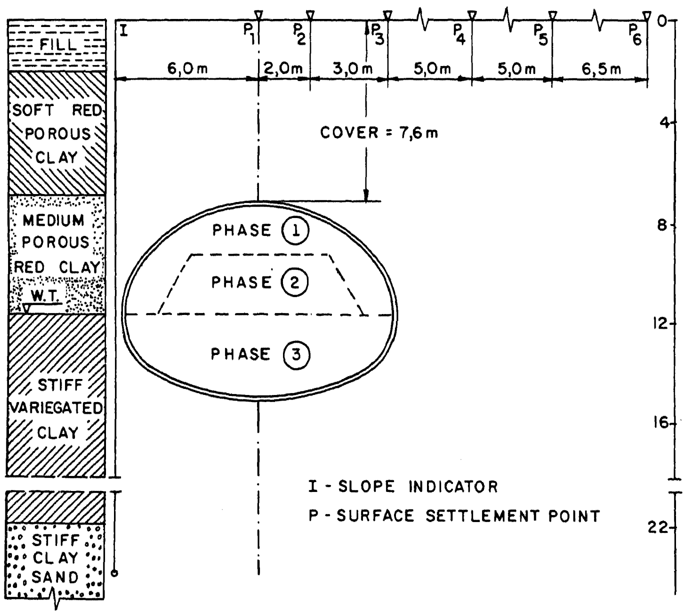

Fig. 3 Geological profile, excavation phases sequence and monitoring schemes

Samples for the laboratory tests were prepared from 50-cm blocks extracted from 3.5 m, 6.5 m (porous red clay) and 9.5 m, 12.5 m (variegated clay) depths. The porous red clay contains, on average, 5% sand, 16-28% silt and 67-79% clay. The bulk unit weight is of the order of 15 kN/m^3. The natural moisture content is, on average, 41% and the void ratio varies from 1.52 to 1.62. The variegated clay contains, on average, 3-10% sand, 1-3% silt and 89-94% clay. The bulk unit weight is of the order of 17.6-18.1 kN/m^3. The natural moisture content is 36% and the void ratio is 1.0, on average.

In order to obtain the stress-strain behavior of these soils, a number of CD (consolidated drained) triaxial tests were carried out on undisturbed samples of the porous red clay (3.5 and 6.5 m) and the variegated clay (9.5 and 12.5 m). Because it is well known that the porous red clay behavior is very sensitive to moisture content variations, this material was tested at the natural moisture and only the variegated clay was saturated by back pressure. Figures 4 to 6 present some of the test results. The Mohr-Coulomb envelope obtained from the investigation shows, for the porous red clay, a cohesion intercept that varies from 35.4 to 39.8 kPa and an angle of friction varying from 23.3^o to 27.2^o. For the variegated clay, these parameters are 66.2 kPa and 25^o; 0 kPa and 30.6^o for consolidation pressures below and above 800 kPa, respectively.

CONSTITUTIVE LAW OF THE MATERIALS

The nonlinear behavior of a tunnel construction is caused by the nonlinear stress-strain characteristics of the soil and by the continuous change of the structure until the tunnel is completed (Wanninger, 1979). Thus, a realistic modeling of the construction process and of the soil behavior must be considered in the numerical calculation of the deformations that will be induced by tunneling.

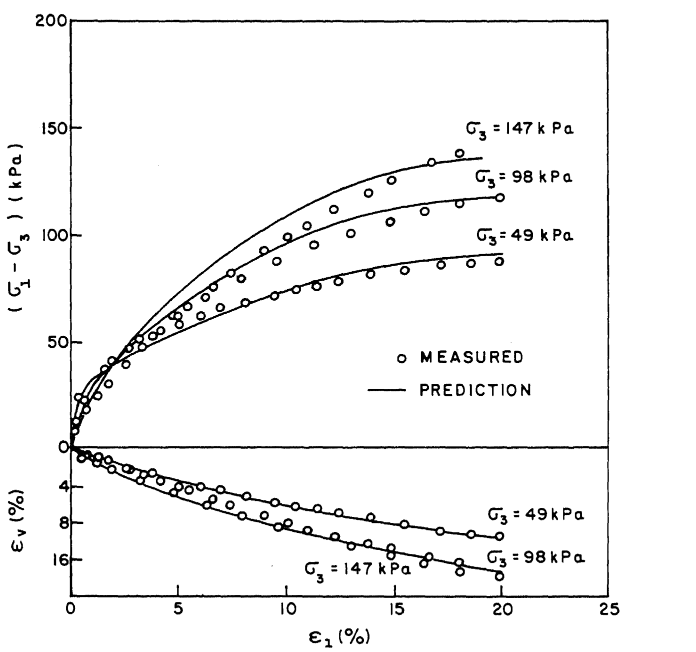

Fig. 4 Comparison of measured and predicted stress-strain relation in CTC tests (3.5 m porous red clay)

The elasto-plastic model developed by Lade (1977, 1979) was used to represent the stress-strain behavior of the soils. This model consists on a model with two yield surfaces, in which the total strain increments are divided into an elastic component, a plastic collapsive component and a plastic expansive component.

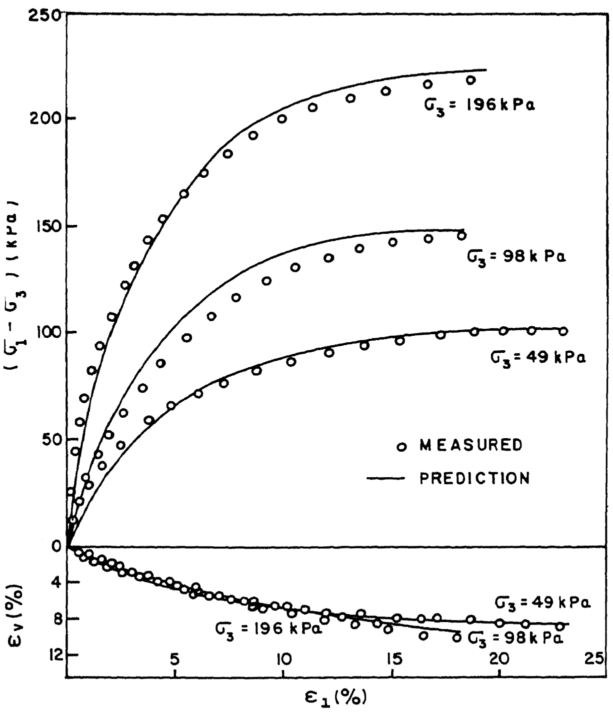

Fig. 5 Comparison of measured and predicted stress-strain relation in CTC tests (6.5 m porous red clay)

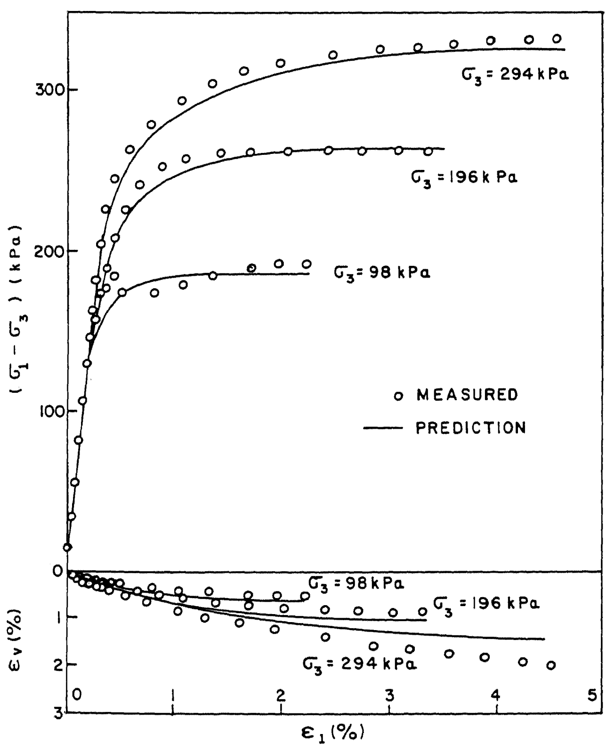

Fig. 6 Comparison of measured and predicted stress-strain relation in CTC tests (9.5 m variegated clay)

Table I shows the model parameters obtained for the red porous clay at 3.5 m and 6.5 m depths; and also for the variegated clay at 9.5 m and 12.5 m depths. For the two last levels, the parameters are the same. Figures 4 to 6 present comparisons between laboratory results and analytical results obtained using Lades's model.

Table I Lade's Model Parameters

		Red-Clay (3.5m)	Red-Clay (6.5m)	Variegated Clay (9.5/12.5m)
Elastic	Kur	98.5	154.0	1526.0
	n	-0.15	0.57	0.61
	ν	0.27	0.27	0.17
Collapsive	P	0.0387	0.02	0.00408
Work-Hard.	c	-	-	-
Failure	η_1	138.4	133.6	254.8
Envelope	m	1.24	0.88	0.992
Expansive	s_1	0.30	0.41	1.35
Plastic	s_2	0.19	0.0	-0.16
Potential	t_1	31.4	3.7	-253.6
	t_2	-58.7	17.5	45.2
Expansive	p	0.4	0.414	0.043
Work-	l	0.37	0.446	1.88
Hardening	α	0.9	1.65	6.11
	β	0.0	0.0	-0.7

FINITE ELEMENT ANALYSIS

The numerical simulation was performed using the geotechnical finite element program ANLOG (Zornberger, 1989). This program, written in FORTRAN 77, was developed for elasto-plastic analysis in plane strain or axisymmetric conditions and uses eight node isoparametric elements. The soil model proposed by Lade (1977, 1979) is incorporated in the program.

The excavation is simulated by stepwise decrease of the balance forces determined at the tunnel perimeter. To calculate these forces, the program uses the procedure proposed by Mana (1978). Excavated elements are automatically removed from the mesh with its subsequent renumbering.

Due the excavation symmetry of the tunnel's face, only half of the tunnel cross section was analyzed . The finite element mesh used contains 154 elements and 503 nodes (Figure 7).

The behavior of the surrounding ground in tunneling is strongly influenced by the advance of the tunnel face and by the construction process, consisting in a essentially three-dimensional problem. Although desirable, a fully three-dimensional analysis of tunnel excavations is out of reach for the majority of the projects due to the computing costs.

The displacements induced during the excavation of a tunnel are produced by two mechanisms: stress relief and stress transfer. In NATM, a significative portion of the displacements is caused by stress transfer before the excavation of the top heading reaches the instrumentation section. In the case analyzed, the superficial settlement on the tunnel's axis due to stress transfer was 38 mm compared to the 80 mm final settlement (Figure 8). Parreira (1991) proposed a simulation for the tunnel construction that considers this fact, allows to calculate when the installation of the support should be done and accounts for three-dimensional effects in two-dimensional plane strain analysis. The analysis presented in this paper was performed using this technique.

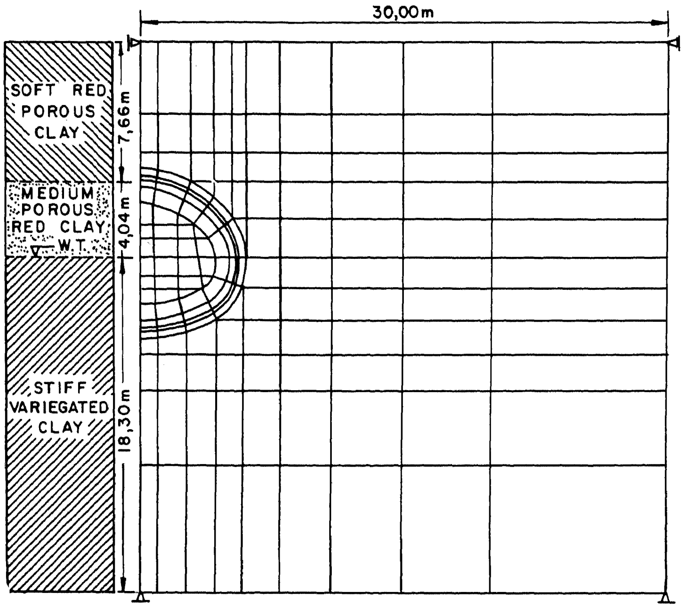

Fig. 7 Finite element mesh

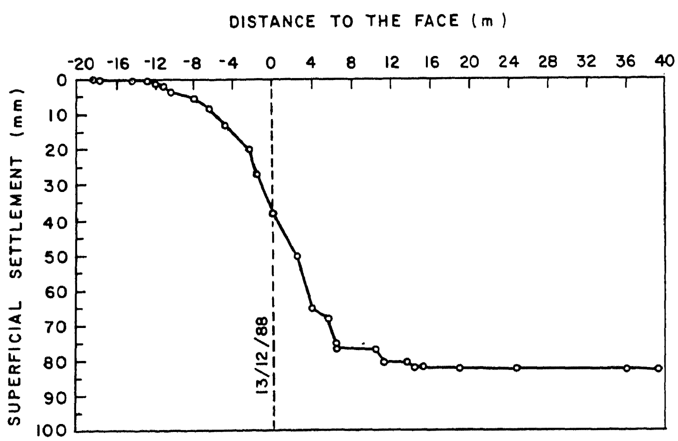

Fig. 8 Superficial settlement x advance of the tunnel face

A series of preliminary post-construction analyses was performed to evaluate the influence of the liner stiffness and the value of the earth-pressure coefficient at rest (K0). These analyses concluded that the liner stiffness had a minor influence on the ground movements, whereas, as expected, the initial state of stress affected very much the displacements. The final analysis presented here was carried out using K0 = 0.38. Details of these preliminary studies may be found elsewhere (Parreira and Azevedo, 1992).

Figures 9 and 10 present the superficial settlements and lateral movements measured by the field instrumentation and obtained by the numerical simulation. The comparison of the observed and calculated surface settlements and lateral movements shows that Lade's elasto-plastic model is able to reproduce the field measurements. The discrepancies observed in the lateral displacements below approximately 11 m may be attributed to errors in the instrumentation measurements - the slope indicator tube was slightly touched during the tunnel excavation.

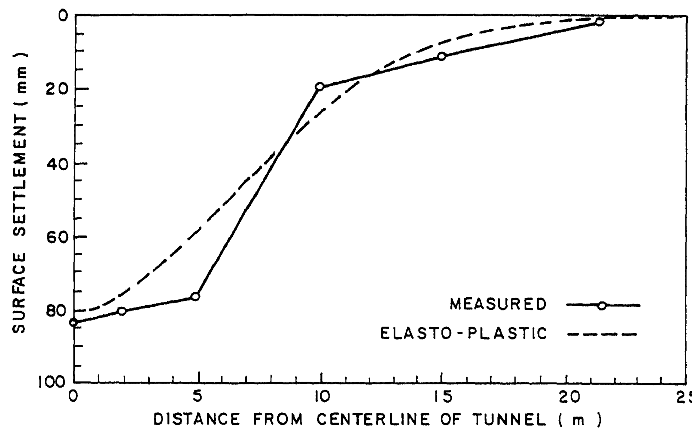

Fig. 9 Comparison of analytical results wit measured superficial settlements

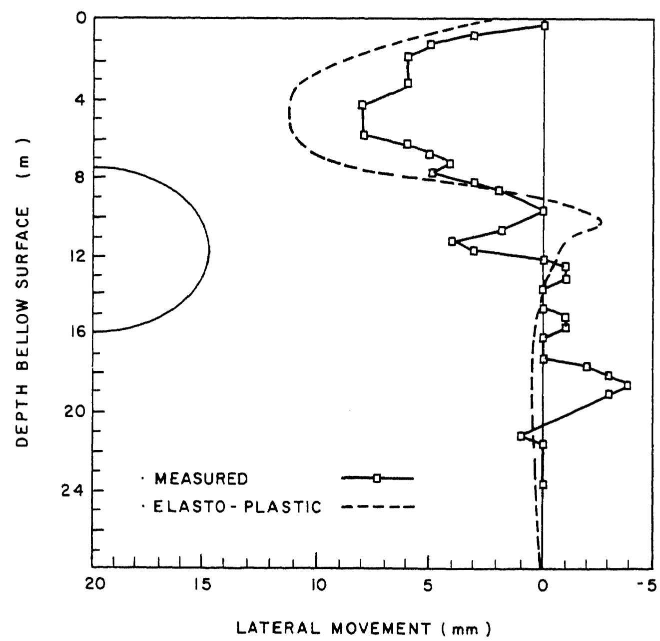

Fig. 10 Comparison of analytical results wit measured lateral displacements

Figure 11 shows curves of same stress rati (SF = h_1/fp) at the end of the excavation. local failure may observed at the bottom of tl tunnel. However, the global stability of tl tunnel is not affected.

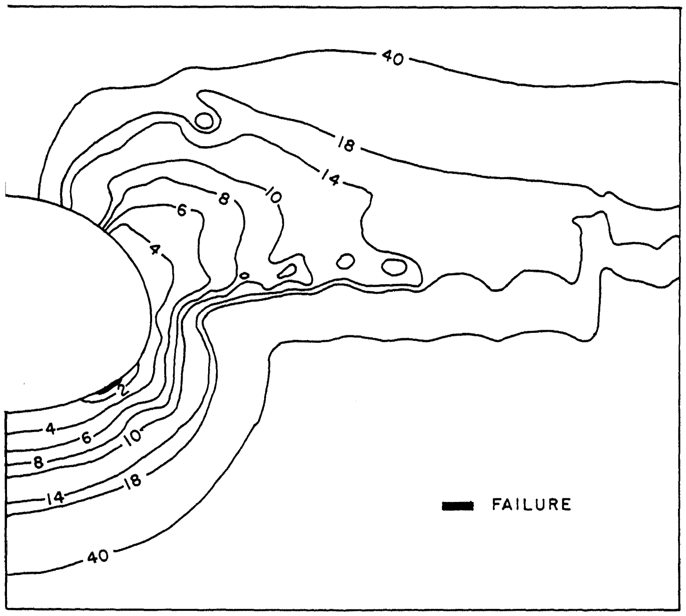

Fig. 11 Isostress-ratio curves

CONCLUSIONS

The elasto-plastic model used was able to reproduce the laboratory behavior of the soils involved in the tunnel excavation. Due to the agreement between the calculated and measured displacements, it may be concluded that the tunneling simulation method used in this paper is adequate to predict displacements surrounding tunnels with thin cover in soft-ground.

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