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Elasto-plastic finite element analyses of a braced excavation and a tunnel

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ABSTRACT

This paper deals with the numerical analyses of an excavation and a tunnel built during the construction of São Paulo city metropolitan. The excavation was made with a retaining system that consisted of soldier piles with lagging, whereas the tunnel was constructed by the New Australian Tunneling Method (NATM). Initially, the paper presents the computer program developed for the analyses and the geotechnical laboratory properties of the soils involved in the constructions. Subsequently, using the new program and the geotechnical properties, elasto-plastic solutions for the excavation and the tunnel are obtained and some displacements observed by the field instrumentation during the tunnel construction are compared with the analytical results. At the end, some conclusions are drawn, basically mentioning that the elasto-plastic solutions were very promising and compared well with the observed field results.

1. INTRODUCTION

The behavior of excavations (open-cuts or tunnels) have been of considerable interest to geotechnical engineers. In Brazil, as a result of major cities expansion, new urban transportation systems have been built. In particular, the city of São Paulo is presently extending its metropolitan system, which consists of a set of railways running both on surface and underground. Some parts of the underground portion of the lines were built using the cut-and-cover technology, while others, located under intensively developed areas, were built by tunneling.

The construction of these excavations may cause significant surface settlements that can damage nearby buildings. Thus, one requirement on the design of these works is the correct evaluation of the displacements induced in the ground by the excavations. When the excavation is made on soft-ground, the soil behavior is often in the plastic range. In these to the yield function and making the normality criterion to be applied to the soil. A second approach consists on defining a symmetric matrix such as:

$$[D_{ep}{}^{m}] = \frac{1}{2}[[D_{ep}] + [D_{ep}]^{T}]$$

where $[D_{ep}]$ is the real non-symmetric elasto-plastic matrix and $[D_{ep}]^{T}$ is its transpose. This very simple technique turned out very efficient in the analyzed problems, showing rapid convergence. Finally, the third approach consists on assembling the global stiffness matrix only considering the elastic strain components. In this case, the convergence rate proved to be very slow.

In order to define a versatile code that allow for the simulation of different steps of a sequential geotechnical construction, a macro programming language was introduced. The macro programming language is associated with a set of compact subprograms each designed to computed one or more basic steps of a finite element solution process. For example, a specific macro command is used to remove elements from an original finite element mesh to simulate an excavation construction (Clough and Mana, 1976). Bar elements are used to numerically simulate struts and tie-rods. In order to prevent a strut to be submitted to tension forces, a bi-linear constitutive behavior that introduces a minimal bar stiffness at tension was considered. In a similar way, a bar element simulating a tie-rod will not be allowed to yield compression forces. Tie-rods are model by two-node bar elements that connect the wall to the anchor. Normally, this may lead to a great increase in the stiffness matrix bandwidth and hence the storage required. However, this increase in needed storage will be minimal as the skyline scheme was used to store the stiffness matrix. Installation, pre-stressing and removal of the bar elements are all performed with specific macro commands.

The stiffness matrix will be dependent on both the stress-level and the stress-history of the materials. Techniques for solving non-linear stiffness equations are quite widespread. In this work, different incremental-iterative approaches based on the Newton-Raphson scheme were introduced. The stress integration carried out during each equilibrium iteration is performed by using an explicit integration technique, where the solution is obtained by forward integrating over sufficient number of sub-increments in order to obtain the required accuracy. Considering that the model used involves two simultaneous yield surfaces and with the purpose of defining a criterion that allow a physical visualization of the sub-increments magnitude, the number of sub-increments is defined by limiting the maximum strain increment in each integration point.

Appropriate commands allow the data input as well as the creation of plot data files used to produce the displacement and stress fields. A proper macro command is used to situations, elasto-plastic analysis must be used, together with the correct simulation of the excavation process which may have significant influence on the analysis.

The objective of this paper is to present the finite element analyses of a supported excavation and a tunnel built during the construction of São Paulo city metropolitan, using an elasto-plastic constitutive model. Initially, the computer program developed is described. Following, the laboratory work is presented and the elasto-plastic model parameters are obtained. Subsequently, the excavation and the tunnel are analyzed and some results observed by the field instrumentation are compared with the analytical answers. Finally, some conclusions are presented.

2. THE COMPUTER PROGRAM ANLOG

ANLOG is a finite element program developed at PUC-Rio with the objective of analyzing geotechnical problems and, in particular, excavations works (Zornberg, 1989). Two questions were mainly considered in the code development. First, the implementation of Lade (1977) elasto-plastic model to represent the stress-strain behavior of the soil and, second, the appropriate simulation of the sequential steps inherent in the particular constructive process of each problem.

As it is well known, Lade (1977) elasto-plastic model consists on a two yield surfaces model in which the total strain increments are divided into an elastic component, a plastic collapsive component and a plastic expansive component. This last component is calculated by a stress-strain theory that involves a conical yield surface with a non-associated flow rule. This non-associated property of the model have some implications in the finite element implementation. First, non-symmetric solvers are required as the tangential stiffness will be expressed in a non-symmetric form. Second, the necessary machine storage capacity will be nearly doubled. In the developed program, both symmetric and non-symmetric systems of algebraic equations are solved by a direct procedure. Furthermore, the required storage and computational effort are reduced by storing the necessary parts of the upper triangular portion of the stiffness matrix by columns and the lower triangular portion by rows (skyline scheme) (Mondkar and Powell, 1974).

With the purpose of avoiding the difficulties of solving a non-symmetric system of equations, numerical schemes have been developed which enable the use of the symmetric equation solvers in the tangential stiffness program for non-associated materials (Xiong, 1985; Pande and Pietruszczak, 1986). Three different techniques were implemented, besides the non-symmetric one, in order to lead to a symmetric stiffness matrix. While using these approaches, stress integration continues to be performed with the real non-symmetric constitutive matrix. In the first implemented procedure, the real non-associated material is transformed into an equivalent associated material by forcing a few parameters of Lade's model to assume predetermined values, forcing the plastic potential function to be identical solve the finite element problem itself using one of the implemented algorithms according to control parameters supplied by the user. Specification of initial stresses (either geostatic or isotropic) as well as specification of the initial loci of the yield surfaces can also be provided. Other commands are presently available in the code and many other ones could easily be implemented to represent the necessary steps to simulate a specific geotechnical work.

3. THE CASES ANALYZED

In the city of São Paulo, a new subway line with 16.5 km is under construction and will connect to the North-South Line of the existing metropolitan system (Figure 1). One part of this line, named the 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 the NATM and shield tunneling techniques.

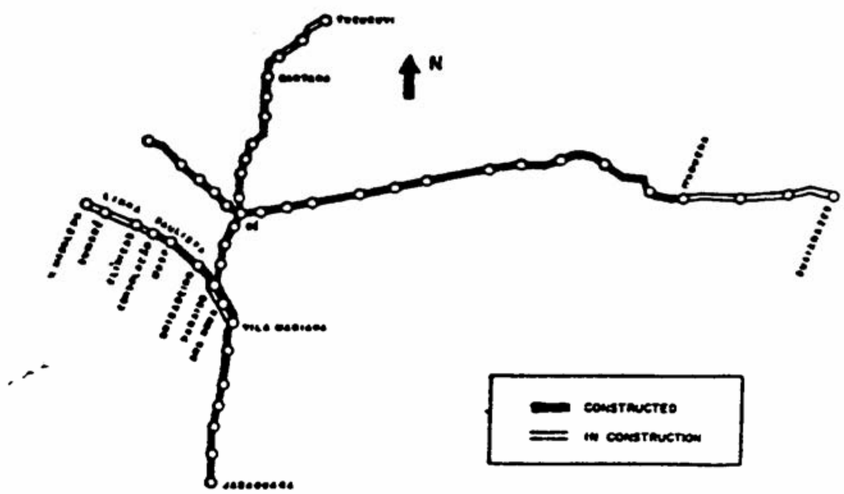


Figure 1 - The São Paulo City Metropolitan System

The subsoil of São Paulo city consists of quaternary and tertiary formations. At Paraíso tunnel, from the ground level downwards, there is a 2m thick layer of fillings. Beneath that, there is a soft to over consolidated stiff porous-silty-clay (STP ranging from 4 to 11), about 9m thick, called São Paulo porous-red-clay. This red clay sits on top of a heavily over consolidated stiff and fissured sand-clay (SPT ranging from 16 to 22), about 11m thick, named variegated clay, which is a tertiary and lateritic material. Bellow the variegated clay there is a very stiff clay-sand (SPT ranging from 28 to 35). The ground-water-table is in frontier between the red clay and the variegated clay, at 12m depth (Figure 2).

In order to obtain the stress-strain behavior of the soils and data for the simulation, a number of triaxial tests were carried out on undisturbed samples of the red clay and the variegated clay (Parreira, 1991). Table 1 shows the parameters of Lade's model obtained for the different materials, whereas Figure 3 to 6 presents comparisons between laboratory results and analytical results obtained using Lade's elasto-plastic model and these parameters.

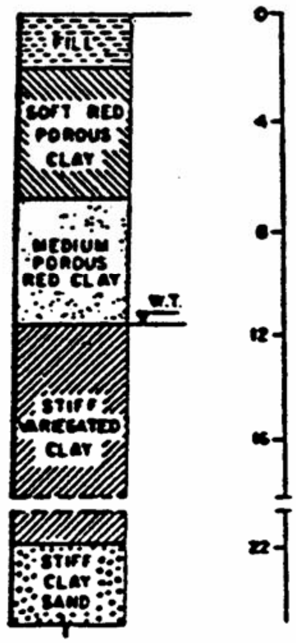
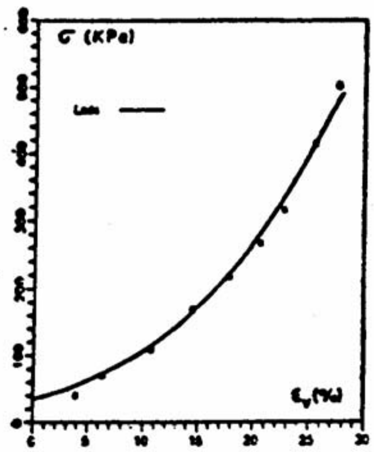


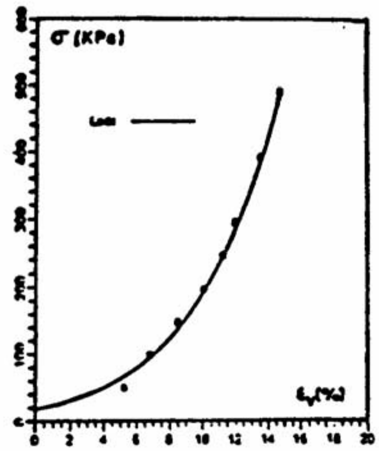
Figure 2 - Geotechnical Profile at Paulista Avenue

TABLE 1 - Lade's Model Parameters

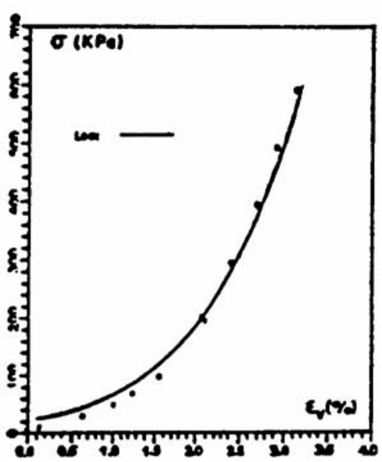
		Red-Clay (3.5m)	Red-Clay (6.5m)	Variegated-Clay (9.5/12.5m)
Elastic	K_{ur}	98.5	154.	1526.
	n	-0.15	0.57	0.61
	v	0.27	0.27	0.17
Collapsive Work-Hardening	P	0.0387	0.02	0.00408
	c	-	-	-
Failure Envelope	η_1	138.4	133.6	254.8
	m	1.24	0.88	0.992
Expansive Plastic Potential	s_1	0.3	0.41	1.35
	s_2	0.19	0	-0.16
	t_1	31.4	3.7	-253.6
	t_2	-58.7	-17.5	45.2
Expansive Work-Hardening	p	0.4	0.414	0.043
	l	0.37	0.446	1.88
	α	0.9	1.65	6.11
	β	0	0	-0.7



(a) Red Clay (3.5m deep)



(b) Red Clay (6.5m deep)

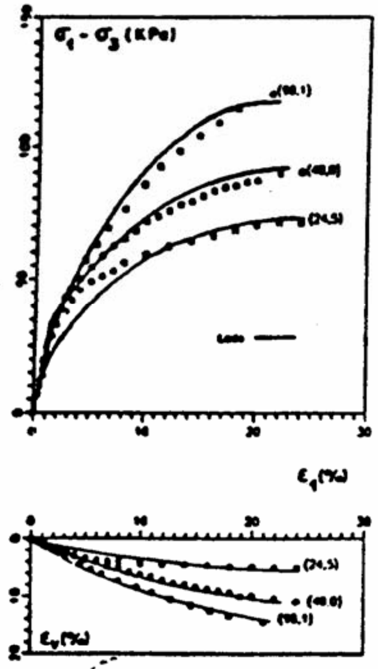


(c) Variegated Clay (9.5/12.5m deep)

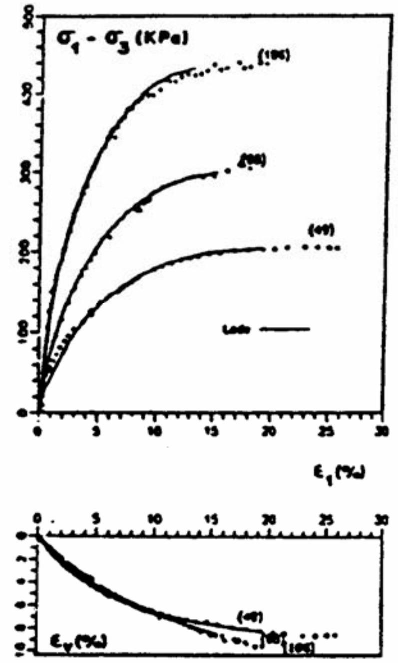
Figure 3 - Isotropic Compression Tests

3.1 The Poço Salas Técnicas Excavation

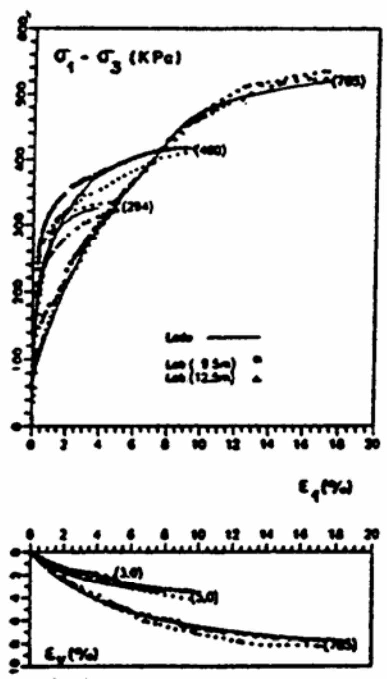
The Poço Salas Técnicas is located near Consolação Station (Figure 1) and consists of an excavation 20.8m long, 16.2m wide and 31m deep (Zornberg, 1989). It was located adjacent to a 17 floors building with 2 underground levels. Naturally, this building deserved a special consideration due to the possible settlements that could occur as a consequence of the excavation (Figure 7).



(a) Red Clay (3.5m deep)

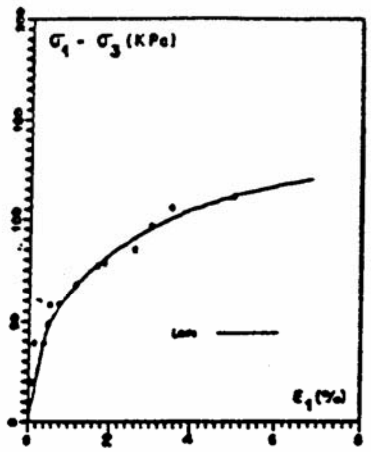


(b) Red Clay (6.5m deep)

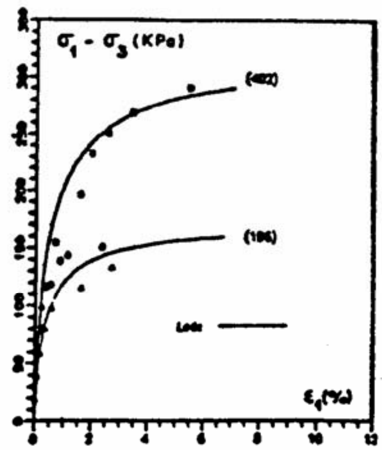


(c) Variegated Clay (9.5/12.5m deep)

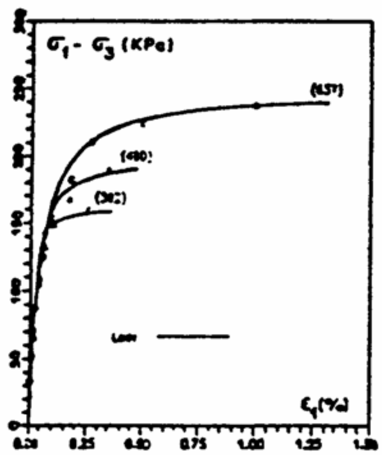
Figure 4 - Drained Conventional Triaxial Compression Tests



(a) Red Clay (3.5m deep)



(b) Red Clay (6.5m deep)



(c) Variegated Clay (9.5m deep)

Figure 5 - Drained Reduced Triaxial Compression Tests

The excavation retaining structure consisted of a system of soldier piles with wood lagging and three strut levels with no pre-stressing forces. The water table was initially lowered from its initial level to, approximately, 4m below the future excavation bottom (Figure 8).

The finite element mesh consisted of 481 nodal points and 147 elements, 3 of which were bar elements and the rest were 8-node isoparametric elements (Figure 9). The initial state of stresses was characterized by a geostatic state defined from the knowledge of the at rest earth pressure coefficient for each material. The first analysis consisted on simulating

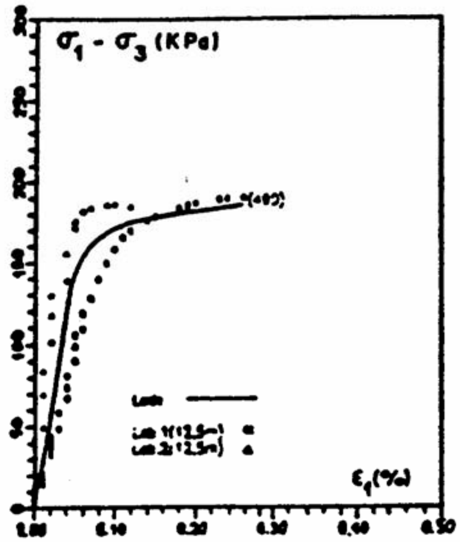


Figure 6 - Drained Reduced Triaxial Extension Test. Variegated Clay (12.5m deep)

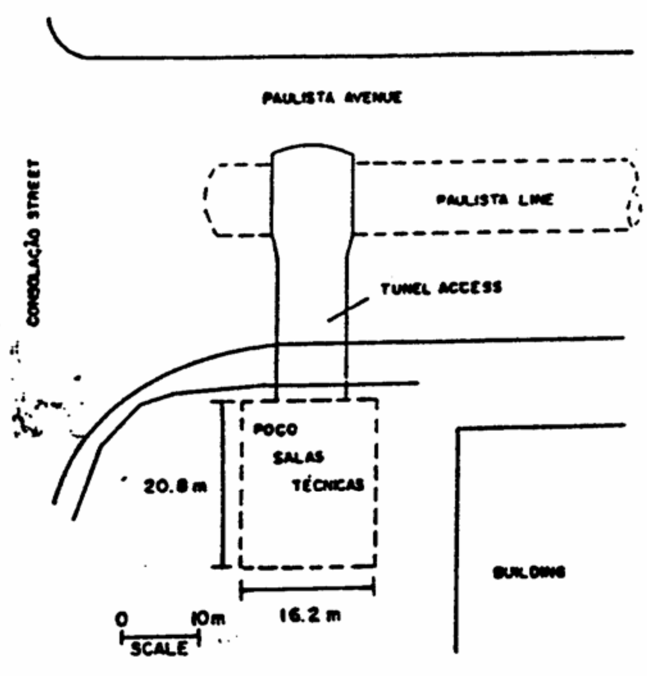


Figure 7 - Localization of the Poço Salas Técnicas Excavation

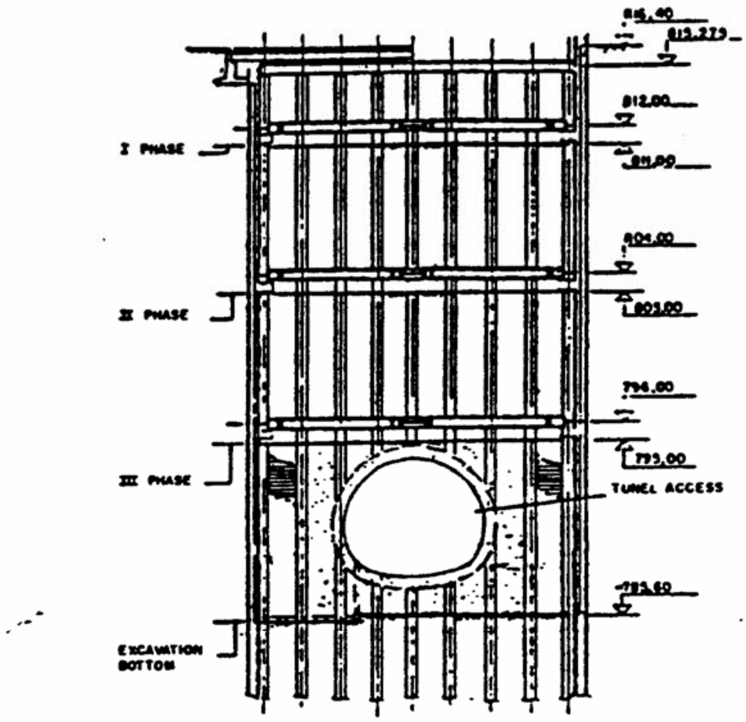


Figure 8 - Cross-Section of the Poço Salas Técnicas Excavation

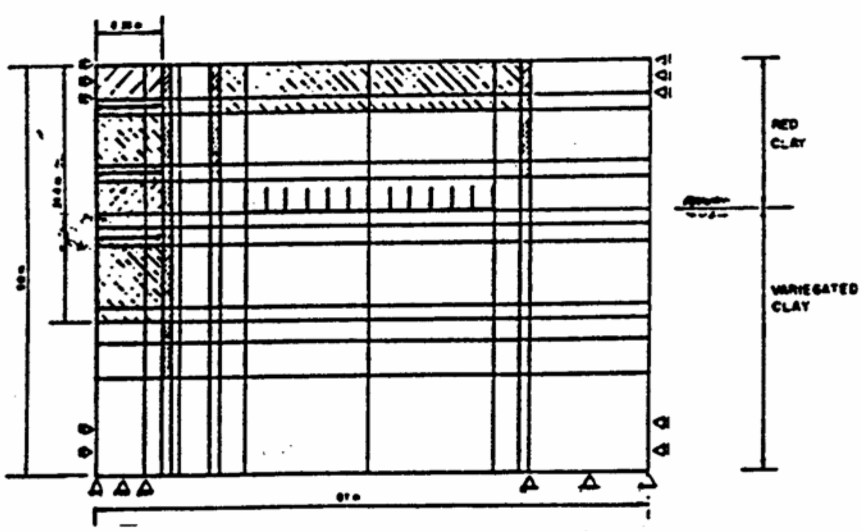
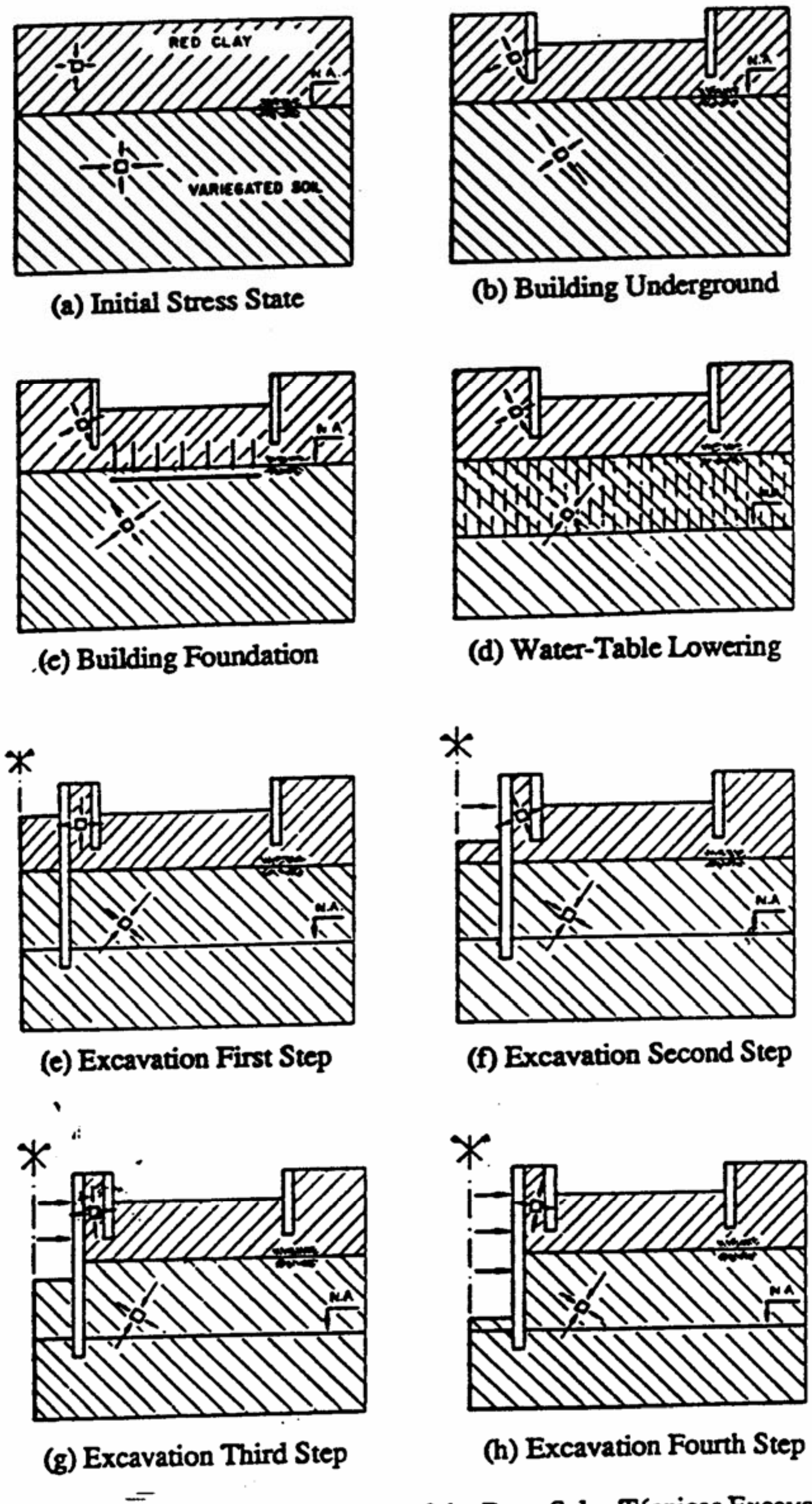


Figure 9 - Finite Element Mesh of the Poço Salas Técnicas Excavation



(a) Initial Stress State

(b) Building Underground

(c) Building Foundation

(d) Water-Table Lowering

(e) Excavation First Step

(f) Excavation Second Step

(g) Excavation Third Step

(h) Excavation Fourth Step

Figure 10 - Finite Element Analyses of the Poço Salas Técnicas Excavation

the excavation of the two underground floors of the building. The loading applied by the building foundation was further represented numerically. Afterwards, the stress field was obtained by simulating the lowering of the water-table, getting the state of stress in the soil mass previous to the start of the excavation construction. Four finite element analyses, corresponding to the each constructive phase of the excavation execution were subsequently developed. The struts installation was performed in the corresponding stages of the simulation. These analyses were carried out using the Standard Newton-Raphson Method, considering either the non-symmetric or the symmetric stiffness matrix during the non-linear solution (Figure 10).

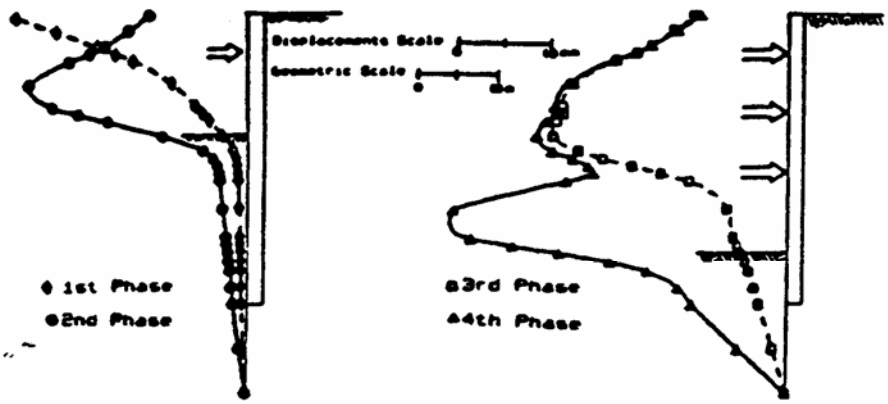


Figure 11 - Horizontal Movements of the Retaining Wall

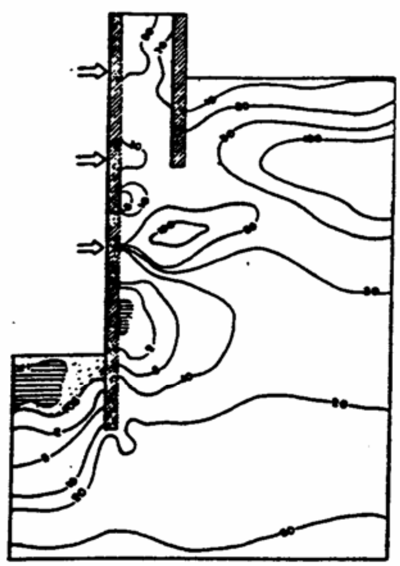


Figure 12 - Isostress-ratio Curves .

The stress states, displacements fields and safety factors of the soil mass, as well as the efforts and the different components of the retaining structure, were obtained from these analyses. The minimal settlements of the adjacent building obtained numerically were confirmed with the insignificant values measured in the field. The wall displacements during the four constructive excavation phases can be observed in Figure 11. These displacements are influenced by the high force exerted by the last strut level. The maximum lateral displacement calculated (3.69cm) is in coincidence with the development of a localized active failure area in the soil mass adjacent to the wall.

Figure 12 shows curves of same stress ratio (SR = η_1 / fp) at the end of the excavation. It can be observed that at the excavation bottom and behind the retaining wall, between the last strut level and the excavation bottom, local failure regions occurred. Globally, however, the excavation was safe.

3.2 The ParaísoTunnel

The Paraíso tunnel is located near the Paraíso Station and was built using NATM. This tunnel, approximately 103m in length, is characterized by a low depth cover, ranging from a minimum of 7.5m to a maximum of 9m (Parreira, 1991). At the instrumented section, the ground cover was about 7.6m. The tunnel cross-section is non-circular, 8.4m high and 11.4m wide (82 m^2 net area), with primary and secondary shot-concrete support (20cm and 15cm thick, respectively), without invert (Figure 13).

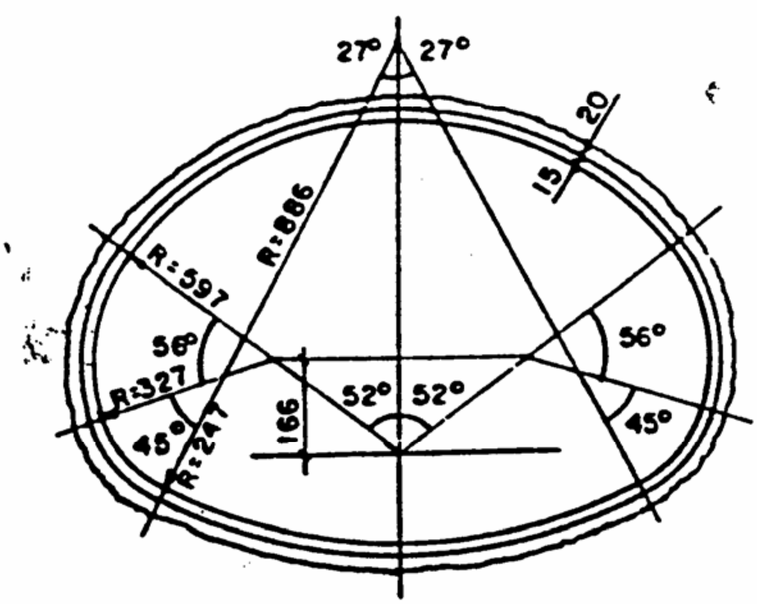


Figure 13 - The Paraíso Tunnel Cross-Section

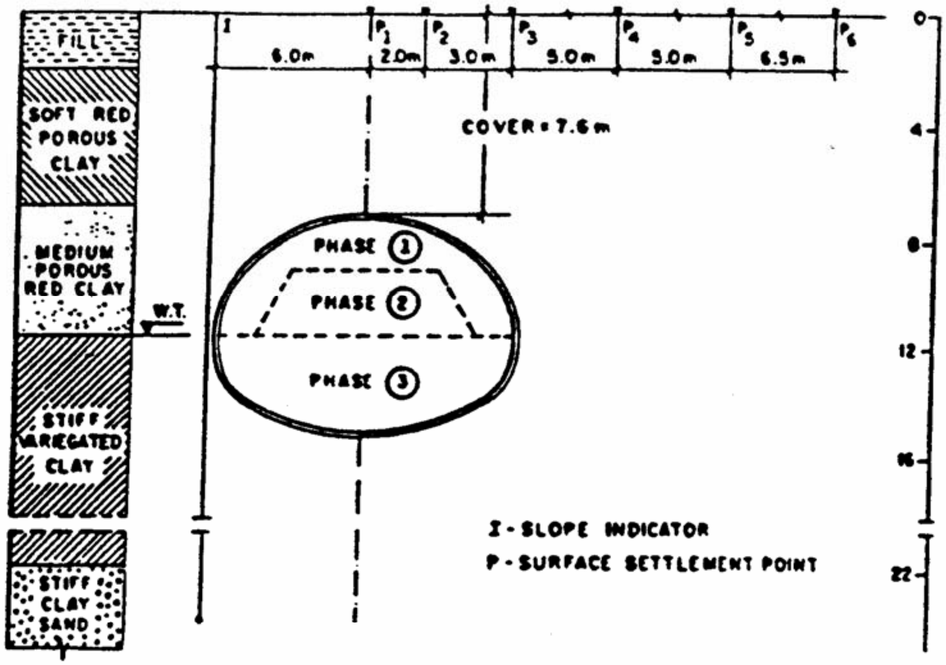


Figure 14 - The Paraíso Tunnel Construction Scheme and Instrumentation

The top half of the tunnel cross-section was driven through the red clay and the rest through the variegated clay (Figure 14).

- Tunneling Procedures and Field Instrumentation

The tunnel construction was carried out by heading excavation and providing primary support consisting of L-shaped steel ribs (127mm) and shot-concrete 20cm thick (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 1.6m segments. Similar procedures were continued in the bench tunneling (phase 3) which was performed 7m behind the heading face. After the tunnel completion, the shot-concrete lining thickness was increased to 35cm.

To monitor the ground behavior during tunneling, one slope indicator, 6 surface settlements points, 5 convergence pins and 2 multipoint vertical extensometers were installed. Only the readings obtained with the slope indicator and the surface settlement points were sufficiently consistent to be used for the purposes of this paper (Figure 14).

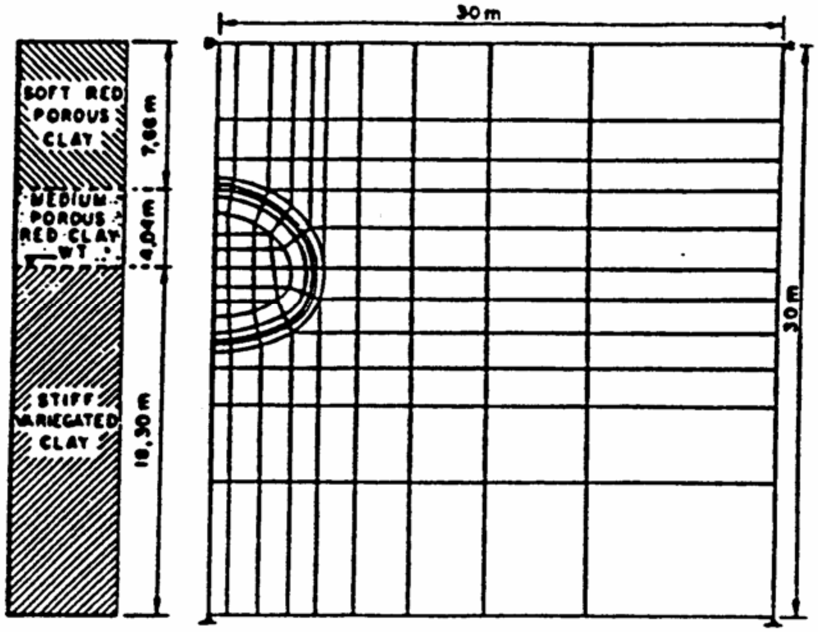


Figure 15 - The Paraíso Tunnel Finite Element Mesh

- Tunneling Simulation

Only half of the tunnel cross-section was analyzed, due to symmetry of the tunnel's face excavation. The finite element mesh used is shown in Figure 15. The number of elements is 154 and the number of nodal is 503.

It is well known that the behavior of the surrounding ground is very much influenced by the advance of the tunnel face and by the construction process which is, essentially, three-dimensional. Although desirable, a fully 3-D analysis of tunnel excavations is out of reach for the majority of the projects due to the computational efforts and costs. Thus, several methods which account for the 3-D effects in a 2-D plane strain analysis have been proposed by many authors (Parreira, 1991).

The displacements induced in the ground during the excavation of a tunnel are due to two mechanisms: stress relief and stress transfer. In NATM, the greatest displacements occur by stress transfer, before the head of the excavation reaches the instrumented section. This paper proposes a simulation for the tunneling construction that considers this fact and also allows to calculate when the support installation should be made.

The method proposed by Parreira (1991) involves the following four stages (Figure 16):

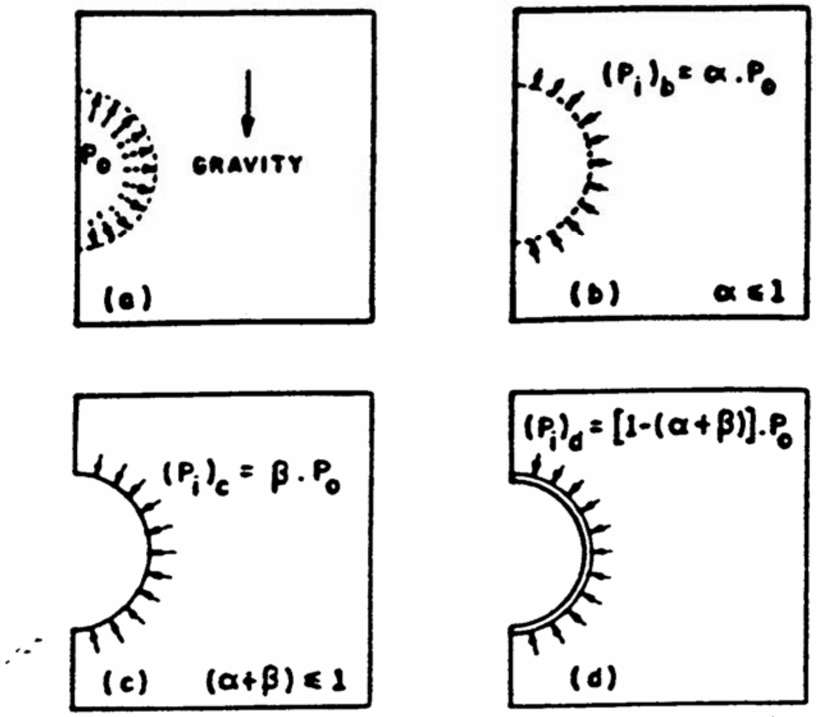


Figure 16 - Tunnel Simulation Stages

1) The initial stress state in the ground (Figure 16a) is assumed to be at Ko condition and the nodal forces (-Po) caused by the excavation at the tunnel surface are calculated.

2) In order to obtain the displacements that occur before the excavation front reaches the instrumented section, the stresses at the tunnel cross-section contour are released by applying a percentage of Po (α.Po) necessary to induce a known surface settlement. This known settlement is assumed from prior observations or from a previous numerical simulation of the longitudinal advance of the tunnel (Pierau, 1982; Hanafy and Emery, 1982). The release in this and in the following stages are applied gradually. In the case analyzed in this paper, the observed settlement was 38.2mm, corresponding to 46% of the maximum final settlement. The percentage of release (α) in this stage, considering the elasto-plastic model, was 60% (Figure 16b).

3) To simulate the instrumented section excavation, corresponding elements are eliminated from the mesh and a trial-and-error process is used to determine the percentage of release associated with the tunnel's collapse defined in the numerical analysis by a lack of convergence in the iterative process. Then, a release (β.Po), smaller than the collapse release, is applied at the contour of the section, in such a way that it is ensured that

breakdown of the tunnel will not occur (Figure 16c). In the case analyzed and considering the elasto-plastic model, the collapse release was found to be 29% of Po. Therefore, the value chosen for β was determined using a factor of safety (F) equal to 1.4, that is:

$$\beta = \frac{29\%}{F} = 21\%$$

4) The installation of the liner is simulated by changing the materials characteristics at the tunnel contour into the characteristics of the liner and then applying the remaining release $(1-\alpha-\beta)Po$. In the case studied and considering the elasto-plastic model, this rate was 19% of Po. The shot-concrete was modeled by a linear-elastic model (Figure 16d).

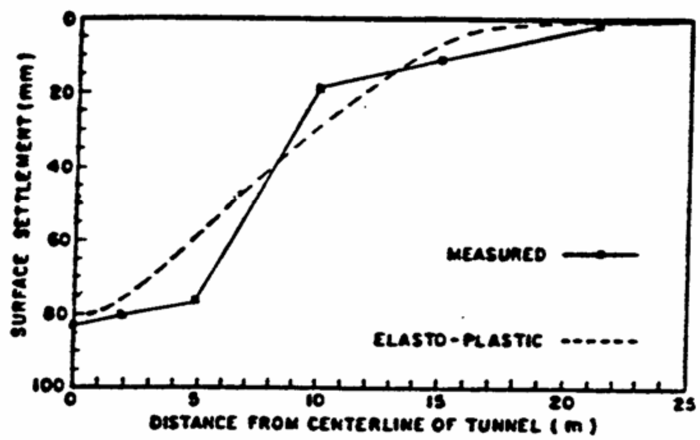


Figure 17 - Comparison between Field and Numerical Vertical Displacements

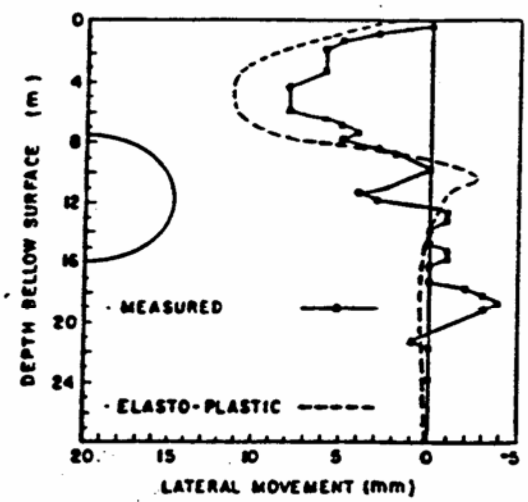


Figure 18 - Comparison between Field and Numerical Horizontal Displacements

Figures 17 and 18 show the superficial settlements and the lateral movements, in this order, measured by the field instrumentation and obtained by the numerical simulation using the elasto-plastic model. These comparisons shows that the numerical analysis was able to reproduce the field values quite accurately.

4. CONCLUSIONS

This paper dealt with the numerical analyses of an excavation and a tunnel built during the construction of São Paulo city metropolitan. The computer program developed for the analyses and the geotechnical laboratory properties of the soils involved in the constructions were presented. Using the new program and the geotechnical properties, elasto-plastic solutions for the excavation and the tunnel were obtained and some displacements observed by the field instrumentation during the tunnel construction were compared with the analytical results. The major conclusions that could be drawn are:

- the elasto-plastic model utilized was able to reproduce the laboratory behavior of the soils involved in the excavation and tunnel studied.

- the excavation analysis gave rise to consistent results. Unfortunately, these results could not be validated by field observations because there was no instrumentation to monitor the excavation construction.

- for the tunnel case, assuming that the agreement between calculated and measured displacements is a criterion to validate the analysis, it may be concluded that the tunneling simulation method proposed is adequate to predict displacements induced in soft-grounds by thin cover tunnels.

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