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SYNOPSIS: This paper analyzes a braced excavation executed during the construction of Rio de Janeiro City subway along a street (Catete Street) in which several old and historical buildings existed and should be preserved. The excavation had an average depth of 13 meters and was executed almost ten years ago in a subsoil in which the principal problems were caused by a 6 meters thick loose sand layer, underlaid by a 4 meter thick silty organic clay layer. Several buildings were damaged and had to be repaired. The objective of this paper is to show that even using the incomplete laboratory and field testing data available at the time the design was made, some of this damage could be predicted and, therefore, possibly prevented. The paper firstly describes the excavation and the available information on the subsoil materials. Subsequently, a finite element analysis is made and the results are correlated to observed damage.

INTRODUCTION

During the construction of the first line of Rio de Janeiro city subway, in Brazil, there was one part that passed below a street in which old and historical buildings were constructed, including one, Palacio do Catete (Catete Palace), that was the President's residence and office while Rio de Janeiro was the capital of the country (1763-1900).

The shallow foundations of these buildings were made at a time in which reinforced concrete was not available in the country and therefore they were made with large stones held together with a mixture of sand and cement.

The subway tunnel was built by a cut-and-cover procedure with retaining walls being used to avoid large ground deformations particularly because the excavation comprised almost all the street width thereby being very close to the building foundations. In spite of the retaining structure the movements caused by the excavation in the backfill cracked almost all the old buildings including the Catete Palace that had to be repaired over many years.

The objective of this paper is to show that even based on the little information available on the deformation characteristics of the subsoil materials, an analysis using the finite element method and considering the most important steps taken during the excavation construction, leads to differential settlements in the backfill that according to allowable settlement recommendations should be avoided in order to prevent cracking.

The paper firstly describes the excavation and the available information on the subsoil materials. Subsequently, this information is used to obtain deformability parameters for the different subsoil layers. Finally, an analysis is made, its results are presented and conclusions are outlined.

DESCRIPTION OF THE FIELD CASE

Part of the Rio de Janeiro city subway first line passes below Catete Street and joins Largo do Machado Station to Catete Station, in an area where several small streets and old buildings exist (Figure 1). A cross-section of this part is shown in Figure 2. The field and laboratory data available corresponding to the stretch between Fereira Viana and Dois de Dezembro streets (Figure 1). In this region, borehole number S-505 was considered to be the most representative and its corresponding geotechnical profile is shown in Figure 3. As can be seen in this figure, the first layer is a fill that has an average thickness of 4 metres, an average blow-count number, N(SPT), of 10 and liquid limit (LL) and plastic index (PI) equal to 20% and 10%, respectively. Underlying this fill, there is a 10 metre thick uniform, fine to medium sand layer in which the first 3 metres is in a dense state, relative density (Dr) between 75% and 85%, and the last 7 metres are in a loose state (Dr between 25% and 35%). These relative density values were obtained correcting the N(SPT) according to the vertical effective pressure (Gibbs and Holtz, 1957 and Teng, 1972).

The organic silty clay layer has a granulometric distribution in which the percentage of particles passing through sieve number 200 varies from 40% to 65%. Average values of LL and PI are 43% and 27%, respectively. The value of N(SPT) varies from 2 to 3 and it has a soft to very soft consistency, with the natural moisture content observed in almost all the samples being greater than the liquid limit. The layer can be considered normally

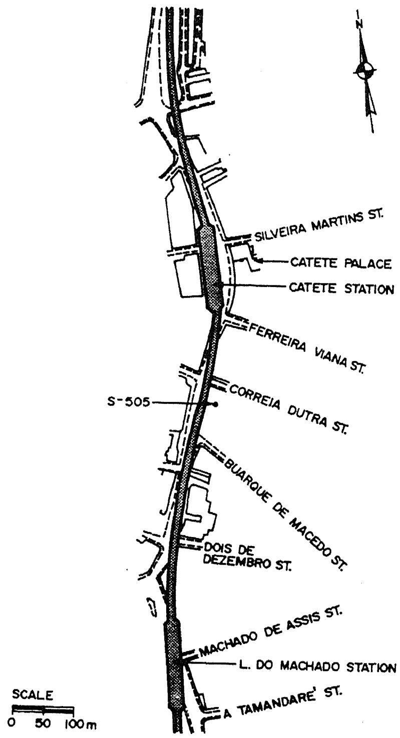

Fig. 1. Rio de Janeiro Subway General View at Catete Street

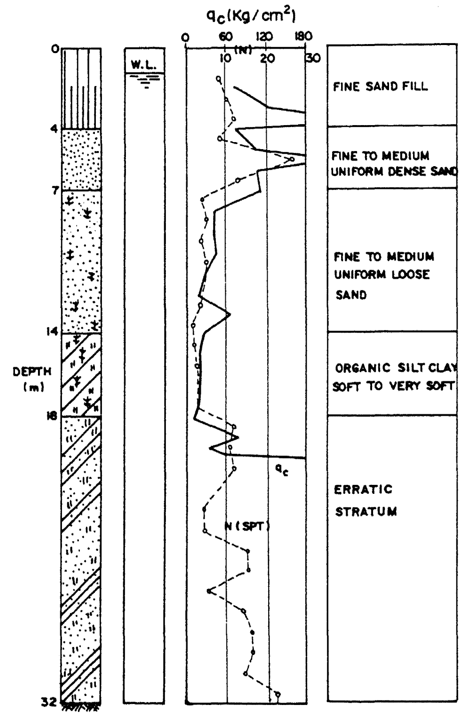

Fig. 3. Geotechnical Profile at Borehole S-505

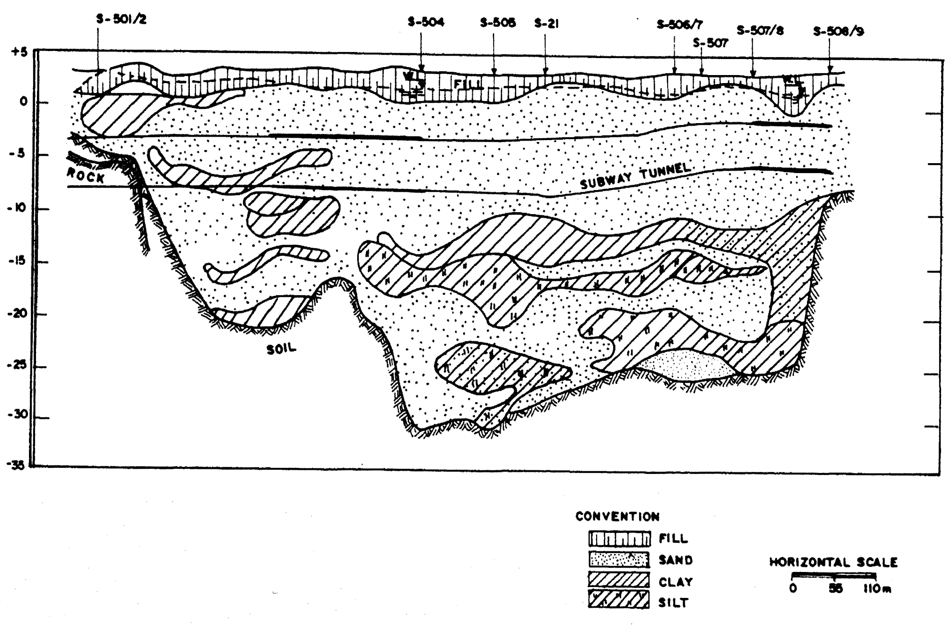

Fig. 2. Rio de Janeiro Subway Cross-Section at Catete Street

consolidated with OCR varying from 1 to 2, and sensitivity being approximately equal to 2. The permeability obtained with conventional oedometer test varied from 10^{-7} to 10^{-9} cm/sy.

The erratic stratum was characterized by having a much higher penetration resistance, average N(SPT) greater than 10. There is little information on this layer but its influence on the excavation behavior is considered to be negligible.

The retaining structure for the excavation slopes consisted of vertical walls supported laterally by three levels of struts that can be seen in Figure 4 where a cross-section and a top view of the retaining structure is presented. Slurry trench (diaphragm) walls incorporated to the final structure of the subway tunnel were used on these retaining structures (Xanthakos, 1979 and Hajnal et al., 1984).

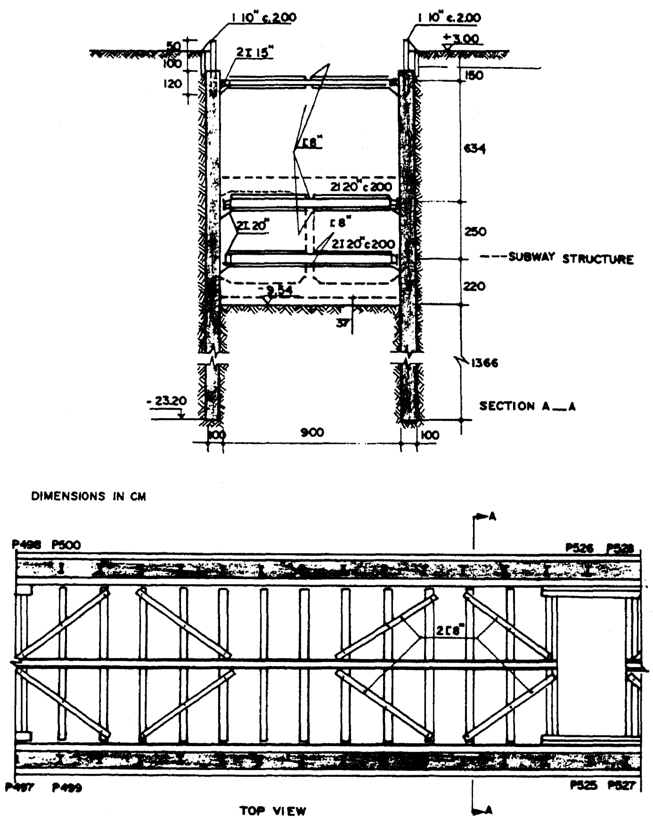

Fig. 4. Rio de Janeiro Subway Retaining Structure at Catete Street

The excavation was executed in four steps as shown in Figure 5. Steps 2, 3 and 4 consisted of first installing struts A, B and C, respectively, and then, excavating.

To control the building movements during excavation, settlement points were installed in most buildings. Figure 6 presents a top view of the location of some of these buildings as well as the observed settlement for different points

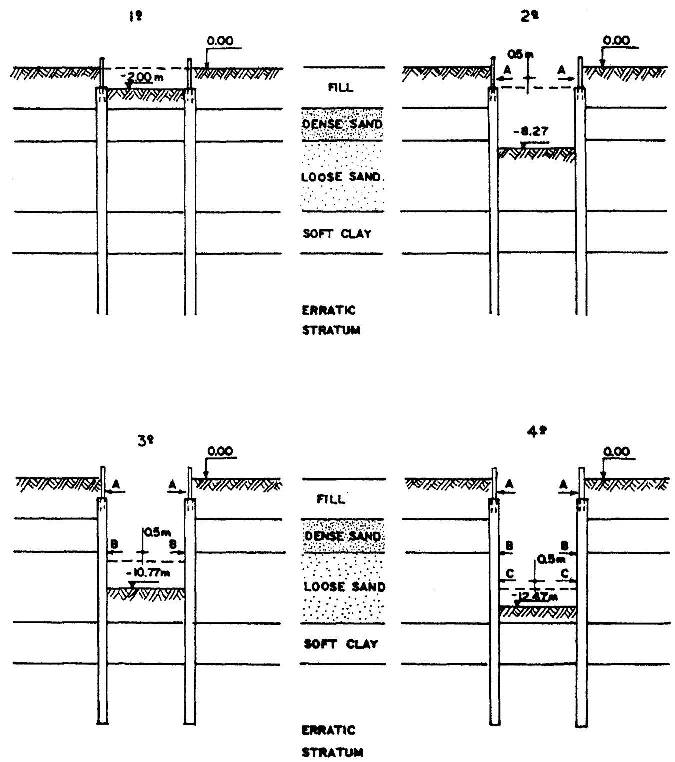

Fig. 5. Excavation Steps

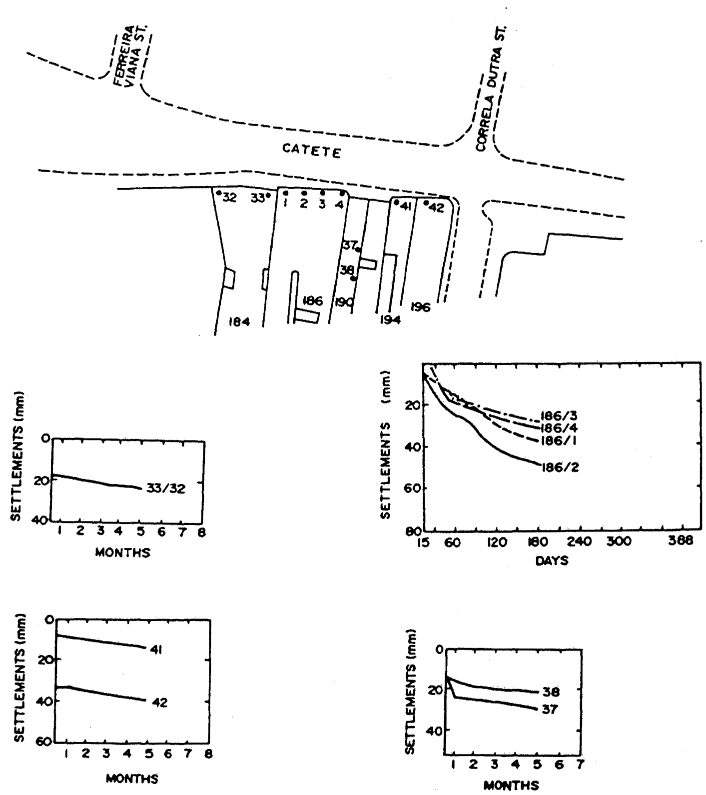

Fig. 6. Settlement of Points in Some Buildings

in each building. Only settlement of building 186 was measured from the beginning of excavation construction. The other points had already moved when the measuring started.

LABORATORY DATA

Besides field penetration tests, some laboratory tests were also performed. Conventional CU (consolidated-undrained) and UU (unconsolidated-undrained) triaxial tests were performed in shelby samples of the organic silty clay layer as well as oedometer and direct shear tests. The sand layer was tested at different relative density with conventional direct shear tests and the erratic stratum was tested with conventional CU tests. No laboratory tests were available for the sand fill material.

Most of these tests did not have consistent results and were not sufficient to find parameters even for the hyperbolic non-linear elastic model (Duncan and Chang, 1970). Therefore they were used to estimate values of elasticity modulus only (Santos, 1980).

NUMERICAL ANALYSIS

The numerical analysis was performed using a finite element program, FENA-2D (Krishnayya, 1973), available at the time. The real problem was idealized as shown in Figure 7 and numerically solved using the mesh presented in Figure 8.

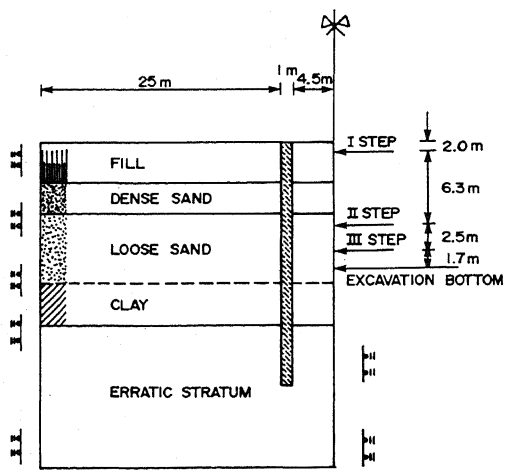

Fig. 7. Problem Idealization

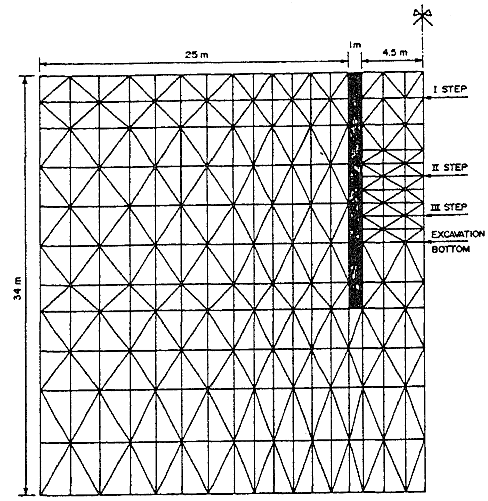

Fig. 8. Finite Element Mesh. 403 Elements and 233 Nodal Points

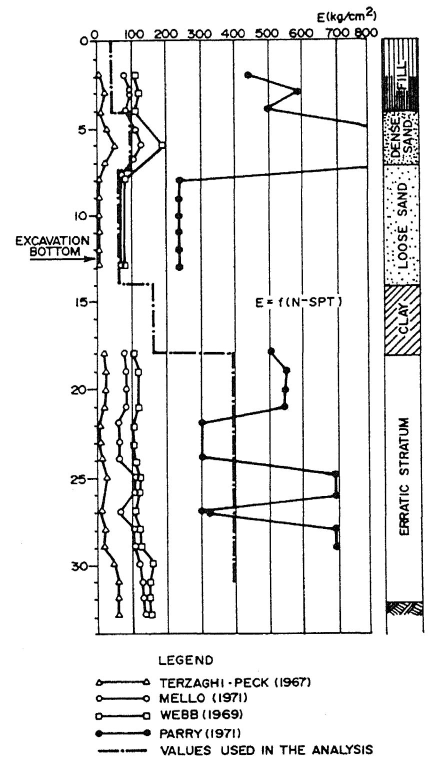

Fig. 9. Elasticity Modulus Correlation

The elasticity moduli (E) used in the analysis were obtained for each layer averaging values obtained with different correlations with N(SPT) (Terzaghi and Peck, 1967; Webb, 1969; Parry, 1971 and Mello, 1971) and the laboratory test results (Figure 9).

Figure 10 shows vertical displacements of surface points near the excavation, together with the lateral movement of the diaphragm wall and the bottom heave, for each excavation step. As can be seen the vertical settlement was much smaller than the ones measured in building number 186 (Figure 6). There are many reasons to explain this difference. First, the use of linear elasticity disregards any decrease

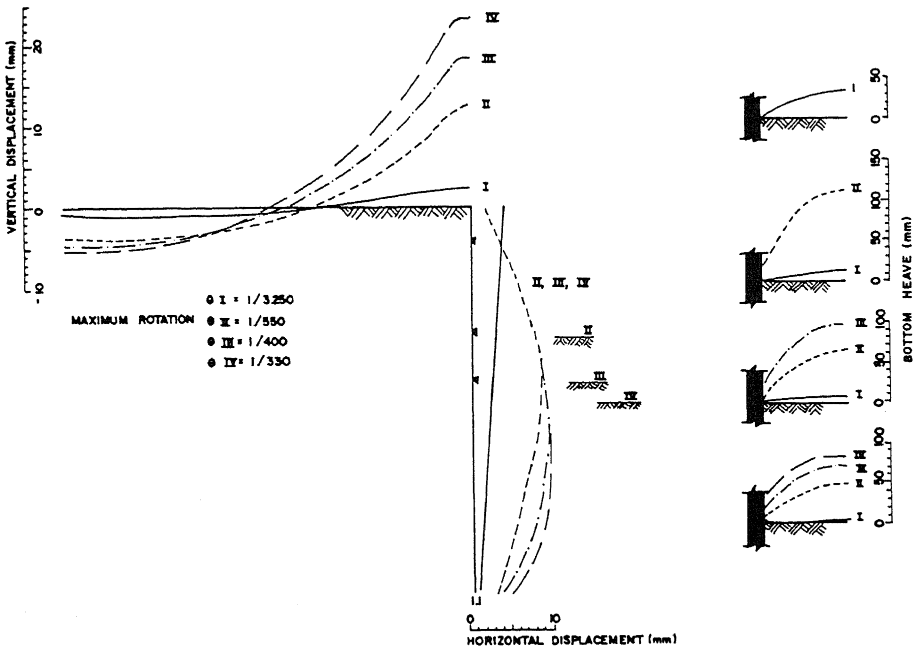

Fig. 10. Analytical (Numerical) Displacements

in the elasticity modulus during increasing shearing, underestimating the settlements. Second, movements caused by lowering the water table level were disregarded. Third, as already shown, the characterization of the subsoil deformability was not properly carried out which could probably lead to smaller settlements and, finally, some assumptions in the numerical model such as rigid struts, boundary and initial conditions might also have led to underestimation of the settlements.

Nevertheless the settlement profile obtained in the analysis gave rise to maximum rotation that may explain the damage which occurred. In fact, according to many authors in different countries (Skempton and MacDonald, 1956; Feld, 1965; Golder, 1971; Grant et al., 1974; Burland and Wroth, 1974; Vargas and Pacheco, 1976 and Novais, 1976) the maximum rotation has to be smaller than 1/300 to avoid wall cracking and to keep windows and doors smoothly functioning. Burland and Wroth (1974) point out that this limit value might be considerably smaller for old buildings and the value of 1/500 is usually used in practice. As is shown in Figure 10 this value is reached in the second excavation step, increasing thereafter. Therefore, these results justify the damage observed in most of the old buildings.

CONCLUSIONS

In this paper an excavation field case associated with the construction of the Rio de Janeiro subway tunnel at Catete Street was reported.

As pointed out by Morgenstern and Eisenstein (1970) to accurately analyze excavations it is necessary to have information on the geotechnical profile, the stress state in the ground prior to excavation, the deformability characteristics of the ground and the construction procedure (boundary conditions).

Unfortunately all this information was not available for the field case reported in this paper, especially with respect to the deformability characteristics of the subsoil. Although some laboratory and field data were available they were not sufficient to find parameters (calibration) for more adequate stress-strain constitutive models. Nevertheless, the paper aimed to use laboratory and field data normally encountered in engineering practice, together with a more accurate characterization of the geotechnical profile, the stress-state in the ground and the construction procedure, to investigate if some of the problems caused by the excavation could have been predicted and therefore possibly avoided.

It was shown that the movements obtained with the numerical analysis, although much smaller than that measured in the field, indeed gave rise to settlement profiles that justify the damage caused in the nearby buildings. Therefore an analysis such as the one presented in this paper would have helped to avoid the damage caused.

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