DEEP EXCAVATION IN SOFT SOILS AND COMPLEX GROUND WATER CONDITIONS IN BOGOTÁ

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INTRODUCTION

A large construction project for a store was completed in December 2004 in the area of the 53th Street and Caracas Avenue in a centric commercial area in the city of Bogotá. The building has two store levels and four parking basementlevels down to 14 m depth, covering 7500 m2 of terrain.

Coluvial deposits with a depth of 26 to 36 m composed of layers of sands, silts and organic clays are found at the site overlying claystones. The ground water level was about 4 m deep with the sand layers having water pressures.

The great construction challenge besides the short time for the construction of the store was to be able to develop a project under the complex geotechnical conditions that are present in the area. Two projects had been attempted previously at the site, in 1971 and in 1988, that had failed because of stability problems with the excavation and construction of the piles. The problems were mainly induced by instability of the soils by water flow.

For the new project a seismic site response study was carried out due to the seismic hazard in the area as part of the additional geotechnical studies. This study provided valuable data on soil stiffness and stress-strain behaviour. Slurry walls enclosing the excavations down to the rock and bored cast-in-place piles were designed for the infrastructure of the building. It was assumed that water conditions would be controlled by the slurry walls and the assumed impervious rock at the bottom of the deposits.

During construction there were some difficulties due to water flow during the excavation of the slurry walls and piles. After the walls were built two pump wells were used to lower the ground water inside the excavation expecting to lower and empty the water underneath the project area. Pumping of water began in early 2004 and a steady flow condition was reached during the almost 12 months of construction indicating incoming flow towards the excavation, either trough the slurry walls or underneath through the rock. A detailed hydrogeologic study was carried out to identify the source of this problem.

Extensive geotechnical instrumentation was put in place to monitor the project. These included vibrating wire piezometers at different levels in locations inside and around outside the excavation reaching the sand layers and underlying rock, inclinometers built in the slurry walls, and settlement benchmarks around the excavation.

GEOTECHNICAL AND HYDRO-GEOLOGICAL CONDITIONS

The lot of the project is located in the piedmont of the hills that surround Bogotá along the east of the city. The deposit has variable thickness between 22 and 36 m, increasing towards the south-west. It is composed of hillside deposits with intercalations of sands, silts, clays and gravels. These strata have water to pressure and they are very unstable in excavations. Under these soils there are rocks of the tertiary Bogotá formation, which are predominantly claystones with locally interbedded sandstone layers less than one meter thick. These rocks were found dipping almost vertically at the site due to a reverse fault that runs along the project area under the deposits parallel to the hills. This fault and the effect on stratigraphy were not identified during the geotechnical studies. Figure 1 shows a geological profile in the east west direction. Due to the stratigraphy and faulting, there was hydraulic continuity of the sandstones at the bottom of the deposit and the recharge areas in the hills.

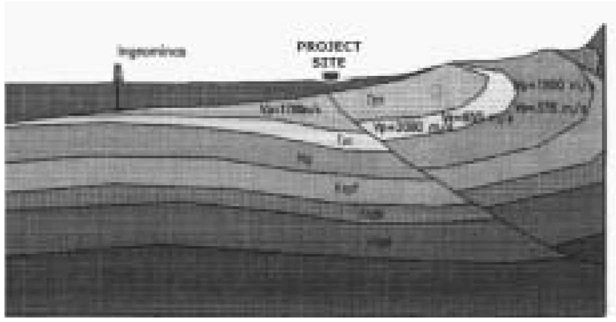

Figure 1: Geological cross section east-west direction at the site. Tpb claystones and sandstones Bogotá formation. Tpc friable sand-stones Cacho formation. Kg, Ksg: claystones and sandstone Guadua and Guadalupe formations.

The geotechnical exploration was developed in several stages at the project site. For the first project in 1971 six (6) borings of variable depth between 25.0 m and 40.0 m were drilled. For the second project in 1988 tree additional borings were conducted to a depth between 27.0 m and 33.0 m, and for the final project two additional borings were made in which Down Hole and Cross Hole tests were taken to determine the shear wave velocity profiles. During construction profile records of all piles and slurry wall modules were kept. This information allowed a detailed description of the soils in the project area. Figure 2 shows a cross section of the project with the identified soil layers, and Figure 3 shows the plan view of the excavation area with the location of sand layers and piezometers.

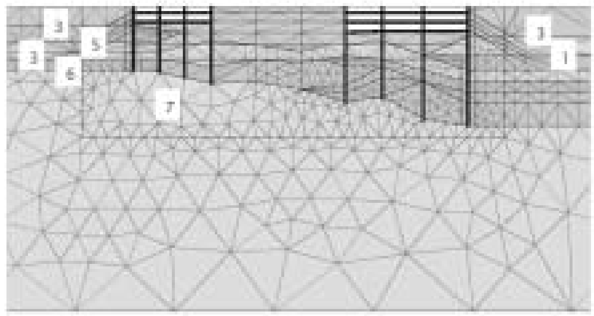

Figure 2: Soil profile east-west section. 1: organic silts, 3: clay, 5,6: sand, 7: rock.

The presence of water bearing soil in the deposits caused the problems with the first two projects at the site. To control this situation the slurry walls were designed and built to act as cut off down to the rock, which was assumed to be impervious. During construction the permanent flow of water pointed out the possibility of water flowing under the walls through the rock. A hydrogeologic model was made based on available hydraulic conductivity data for the different formations and soils, and the stratigraphy at the site. This information was available from well test data. Three 2D models were made using the program PLAXIS. One along an east-west section, and along two north-south sections: one considering the plane of the section in claystones and the other considering it in sandstones. The groundwater boundary conditions were defined based on the levels before the excavation far from the project site, and the pump levels maintained inside the excavation during construction.

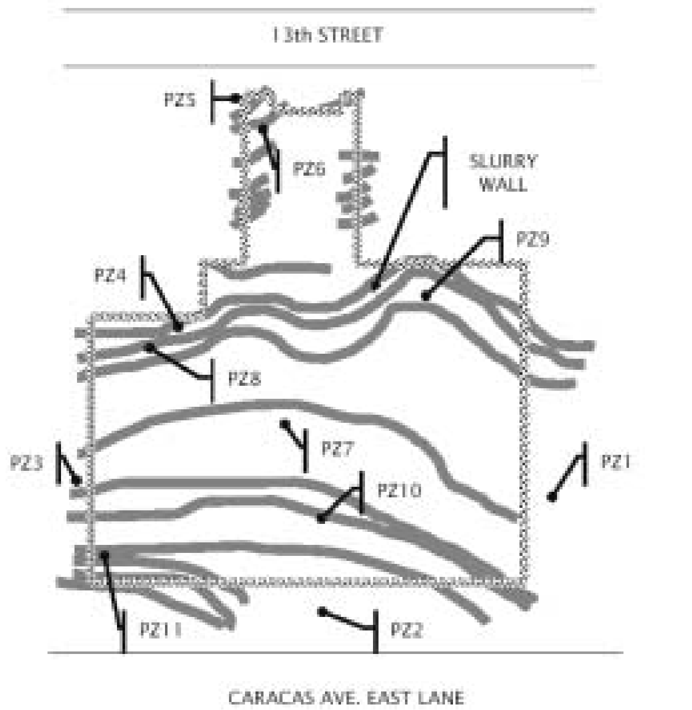

Figure 3: Plan view of the project showing the location of piezometers and sand layers identified during construction underneath the deposits.

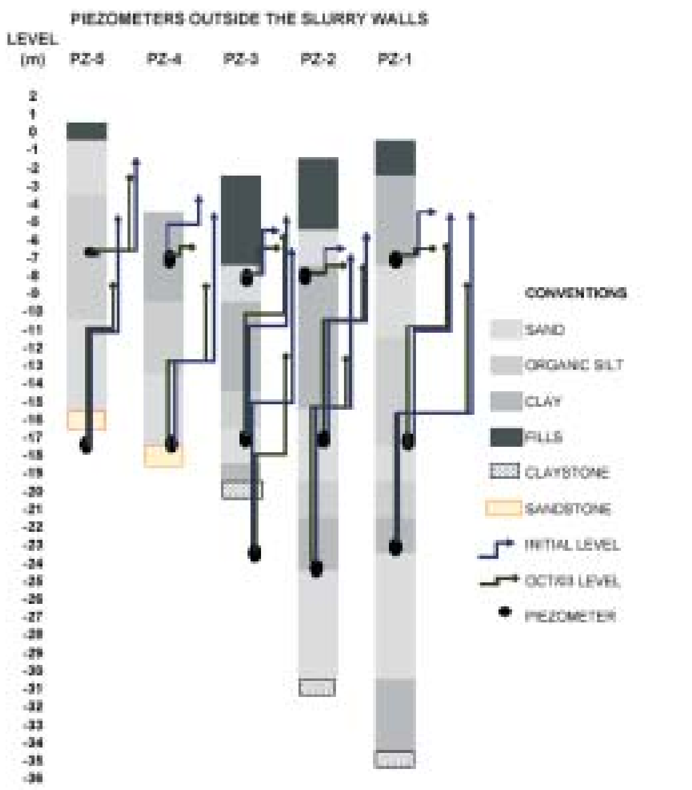

Figure 4: Piezometers data outside the slurry walls

The analyses considered two scenarios. One in which the flow was through the slurry walls, and the other with the slurry walls completely impervious. The groundwater levels obtained from the analyses were com-pared with measured piezometric levels around the excavation. The piezometric measurements are shown in Figure 4. The total discharge considering the thickness of sand layers was compared with the actual discharge measured, which was 80 m3/day. Figure 5 shows a detail of the analyses indicating the effect of the variable permeability on the concentration of flow under the excavation. The areas of concentrated flow coincided with the places where there were difficulties during construction of the piles and slurry wall. The analysis showed that the discharge and water levels that were consistent with the measurements corresponded to the case where the slurry walls were impervious and the flow was under the walls through the sandstone layers. Based on these results the bottom floor of the excavation was redesigned and the slurry walls contractor was released from responsibility of the water flow problem.

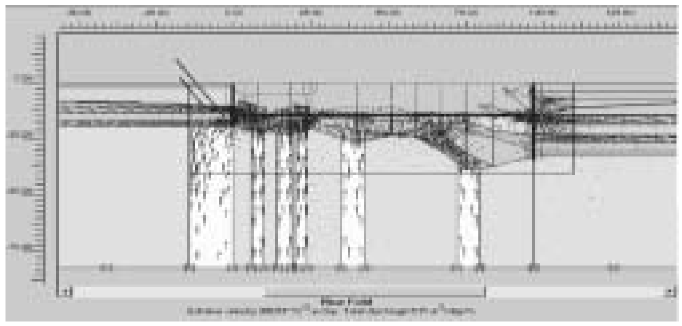

Figure 5: Flow lines north-south model, scenario with flow trough the slurry walls. An important flow concentration is produced trough the sandstones in the underlying rock, and trough sand deposits underneath the excavation. These areas coincided with areas where water flow and difficulties with the construction of piles and slurry walls were encountered during construction.

ANALYSIS OF EXCAVATIONS

The special geotechnical conditions, the existence of nearby buildings and main roads, together with the fact that interferences with existing foundations in the lot should be managed and the stringent time constraints for the project, made the planning and construction of the excavations one of the critical points for the development of the project. To understand and to study what happened during the construction of the excavations of this project, it was sought to simulate the construction of the work by means of a model that involved the different soil materials, the constructive sequence, the existent previous structures, the soil-structure interaction and the variation of the piezometric levels, in a model that kept in consideration the non linear stress strength relationships for the materials. To carry out the analyses a finite elements model was developed using the program PLAXIS. The water flow conditions were obtained from hydrogeologic analyses of the excavations of this same project carried out by Rodríguez et al. (2004b), which were coupled in the model.

The soil model used for the analyses was the "hardening soil" (HS) model of plastic hardening developed by Schanz et al. (1999). This is an advanced model to represent the behaviour of different types of soft and hard soils. The plastic hardening refers to the generation of plastic deformations in the soil due to changes of stress. Two types of plastic hardening are distinguished: one due to shear deformations distinguished by a hyperbolic stress-strain relationship depending on the confining stress (Kondner, 1963, Duncan and Chang, 1970) and another due to volumetric compression. The model also considers a Mohr Coulomb failure envelope, as well as the generation of pore water pressures due to undrained shear.

The parameters for the soils in undrained conditions were obtained by the methodology outlined by Rodriguez et al. (2004a). The stress-strain curves were obtained from the shear wave measurements at very low strains, and available dynamic G/Gmax -vs-deformation curves from the seismic site response study were integrated to define the hyperbolic stress-strain curves required for the model. Figure 6 shows the curve used to model the silty soils.

The analysis considered the following constructive stages:

1 Excavation and demolition of the existent structures in certain sectors of the lot.
2 Construction of the contention slurry wall with anchors with excavation to the level of the first basement (level -3.4 m).
3 Construction of anchors and excavation to the level of the second basement (level -6.8 m).
4 Excavation to the third basement (level –10 m). The wall is supported by the structure already built.
5 Excavation to the fourth basement (level –14 m).
6 This last stage was completed after the building was already in service

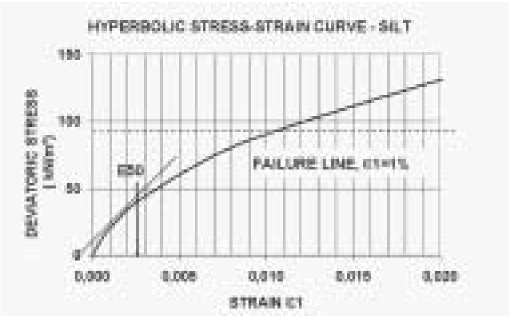

Figure 6: Example of stress-strain curve used for the soils

It should be kept in mind that the analyses were two-dimensional, therefore only traverse sections of the work were analyzed. In the reality the constructive stages in the project planning were not exactly the same as outlined in all cases.

Figures 7, 8 and 9 show measured deformations of the slurry wall for sections in the north and south of the building. Figures 10 and 11 show the settlement of benchmark points located along the west and south sides of the project.

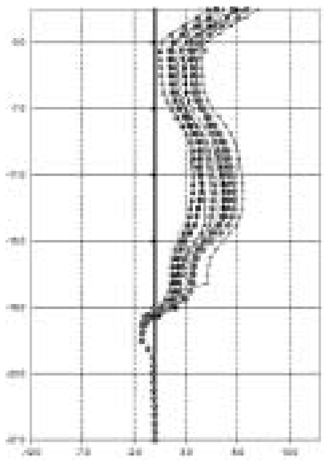

Figure 7: Inclinometer readings, south side wall along 53rd Street. X-axis shows displacement in cm, Y-axis is depth in meters. Several measurements at different dates are shown. Maximum displacement at the end of construction was 8 cm. at -10 to -14 m depths.

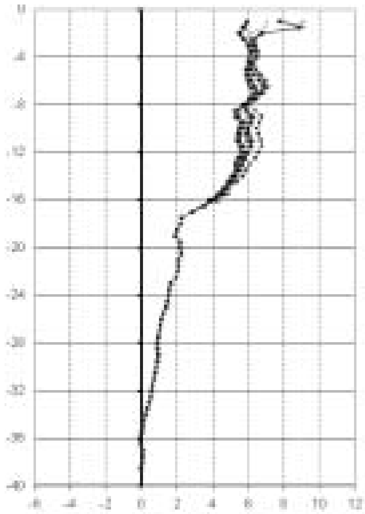

Figure 8: Inclinometer readings, north side wall along 52nd Street. X-axis shows displacement in cm, Y-axis is depth in meters. Several measurements at different dates are shown. Maximum displacement at the end of construction was 6 cm fairly uniform until 12 m depth.

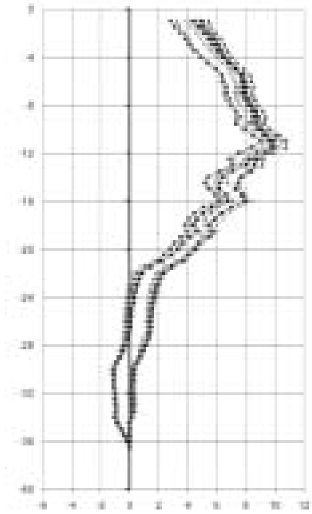

Figure 9: Inclinometer readings, west side wall along Caracas Avenue. X-axis shows displacement in cm, Y-axis is depth in meters. Several measurements at different dates are shown. Maximum displacement at the end of construction was 10 cm with maximum at 12 m depth.

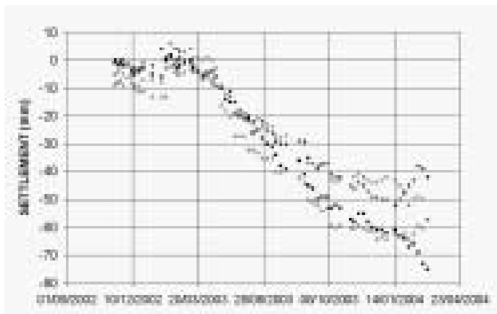

Figure 10: Settlements along Caracas Avenue, west side of project.

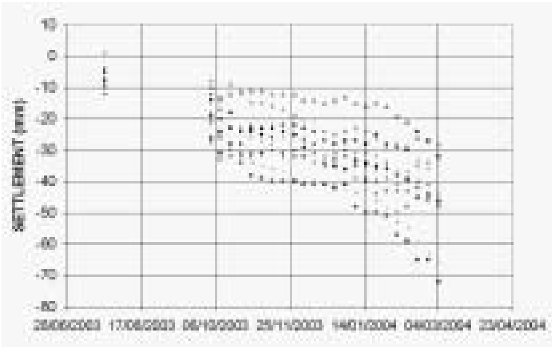

Figure 11: Settlements along 53rd Street, south side of project.

The instrumentation results showed maximum settlements around the excavation of some 6 cm. The settlements varied between 3 and 7 cm, especially along 53rd Street, probably due to local variations in the soil profile. The maximum displacement of the slurry wall was 8 cm towards the bottom of the excavation on the south side, where the depth of the rock layer was 23 m, and 10 cm at 12 m depth on the west side, where the depth of the rock layer is in the order of 33 m.

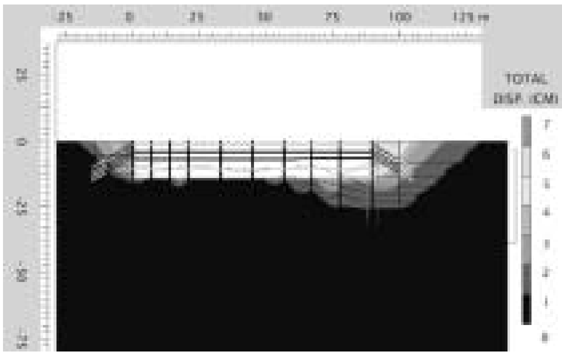

Figure 12: Computed deformations for the north-south cross section at the end of excavation. Maximum wall displacements as well as nearby settlements on the west side are 6 cm. On the east side these values are around 4 cm.

Illustrations of the results of the finite element analyses at the end of construction are shown in Figures 12 and 13. Figure 12 shows contours of computed deformations for a north-south section towards the mid part of the excavation. On both sides of the excavation the depth of the rock layer, and the actual contour of the rock depth are different according to the data obtained from construction records. The rock contact is dipping towards the south west. Therefore the deepest section of the wall is along the Caracas Avenue, where the largest settlements and deformation were measured. The section shown is an intermediate cross-section.

These results show similar trend and similar orders of magnitude of displacements as those measured, both for the north as well as for the south walls. Also the magnitudes of soil settlements computed around the wall are in agreement with the measured data. No attempt was made to try and adjust soil properties to obtain a better fit to the data. These results indicate the capability of the method of analysis used to reasonably predict the behaviour of this complex excavation. It should be noted that other simpler models considered for the problem, particularly those without considering the coupling of groundwater conditions, or considering simplified construction sequences failed to give reasonable results.

The results shown are to the end of construction, at the end of January 2004. The behaviour of the construction is being monitored for ongoing long term deformations, particularly around the building. Although the models used compute excess pore water pressures due to the change in stress conditions, and can do consolidation analysis, no attempts have been made to compute long term settlements. It is foreseen, from other experiences using this approach for projects in Bogotá, that the undrained soil shear stress-strain curves used for the models are not suitable for long term deformation computations.

Figure 10: Settlements along Caracas Avenue, west side of project.

Figure 13. Detail of distribution of computed deformations for the slurry walls north (left in the figure) and south (right in the figure). The deformations pattern and values are consistent with the actual measurements.

CONCLUSIONS

Valuable information has been gathered about the behaviour of slurry walls and the soil anchor system used for the excavation of the project in soft soil conditions with difficult water conditions in the piedmont of Bogotá eastern hills. This is an area undergoing renovation and new developments, with projects similar to the one considered in this paper.

From the analysis of this case it can be concluded that the computational model and the soil models used, considering the coupled problem of deformation and water flow, the highly non-linear behaviour of the soils and the construction sequence, allow detailed study of complex excavations in sectors with especially difficult geotechnical conditions in the short term. The results obtained are in good agreement with the data measured by means of the geotechnical instrumentation. This allows the use of these techniques for future complex projects in the area.

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