Solution for the conclusion of the Mackenzie access in Sao Paulo Line 4 Subway.

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ABSTRACT: The Mackenzie access of Higienópolis Station, of the Sao Paulo Line 4 subway consists of two siccative shafts, one of them was built several years ago and the construction of the second one began in 2012. When 31 m of the second shaft were excavated, it was a soil piping phenomena in the bottom of the shaft. To complete the construction of the Mackenzie Shaft the monitoring data was analyzed by a back analysis to set up the properties of the grown which matched well with the monitoring data. After this, the feasibility of completing the excavation with the protection of jet-grouting piles was verified. The calculations carried out showed that the safety factor was 1.4. In late 2013 it was proceeded to remove the ground stacked in the bottom of the shaft; prior activity to complete the excavation, which was planned in seven sectors. During the excavation of sectors 2, 5 and 7 some problems appeared which were resolved by a punctual drainage. The excavation ended June 24th, 2013.

1 INTRODUCTION

The Station of Higienópolis of the Sao Paulo Line 4 subway has two accesses: Mackenzie and Ouro Preto, as shown in Figure 1.

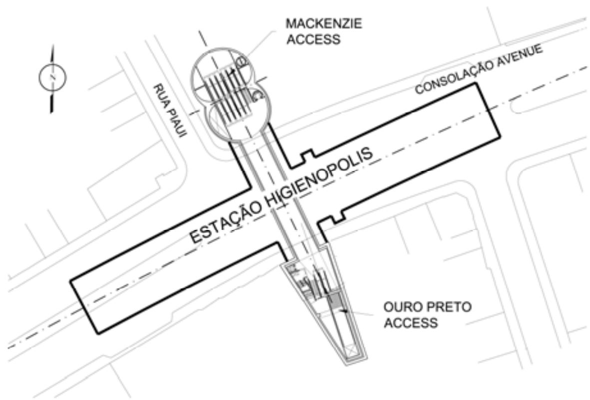


Figure 1. Higienópolis Station and its accesses.

The Mackenzie access is formed by two siccative shafts, with a diameter of 20 m each one. The nearest shaft to the Consolação Avenue was built several years ago, as shown in Picture 1; but the construction of the second shaft, which should have 32 m deep, started in 2012, setting up a vaccum system to go down the water table.

Picture 1. First shaft built in Mackenzie access.

The excavation progressed without difficulties until the depth of 24 m was reached, when it was considered necessary in order to improve the efficiency of the recess of the water table to develop a system of local drains, supported by a vacuum system, as shown in Picture 2.

Picture 2. Drainage pipes with vacuum system.

Saturday, December 8th, 2012, when the bottom of the excavation was at 31 m, a soil piping in the bottom of the shaft appeared, in the area near the vertical of the Piauí Street, which was controlled by stacking about 2 m of ground.

Subsequently to this soil piping, a horizontal crack in the shotcrete which stabilized the shaft occurred, which was repaired, by reinforcing the shotcrete thickness and placing a steel mesh in the following days.

Throughout the months of December 2012 and January 2013 the aim was to improve the behavior of the ground, injecting grout through the shaft; but these injections did not give the expected result so on February 8th it was decided to stop the works until it was defined a solution to end the excavation.

2 ANALYSIS OF THE INSTRUMENTATION

It was carried out a detailed analysis of the measurements available from the instrumentation placed for monitoring the construction of the Mackenzie Shaft, whose implementation is shown in Figure 2.

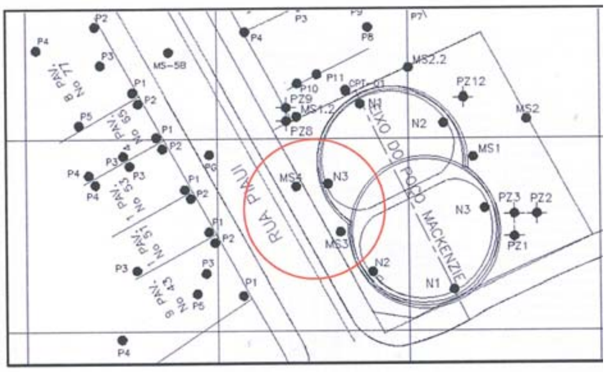


Figure 2. Monitoring for the construction of the Mackenzie shaft.

The monitoring elements are constituted by piezometers (PZ), exterior survey control points (P and MS) and optical measurement points of convergences in the Shaft (N).

The measures which were considered the most representative are those for the survey control points MS3, MS4 and for the topographic checkpoint of the Shaft N3, for being those three in the vertical area where the soil piping occurred in the bottom of the shaft.

Figure 3 shows the evolution of the vertical movements in these three monitoring points and, as it can be seen, after the major changes occurred during the fluidization of December 8th, 2012, the reference points were still descending.

Figure 3 also clearly shows that the greatest movement occurred in the point MS-3, which is the closest to the vertical in the area where the soil piping occurred.

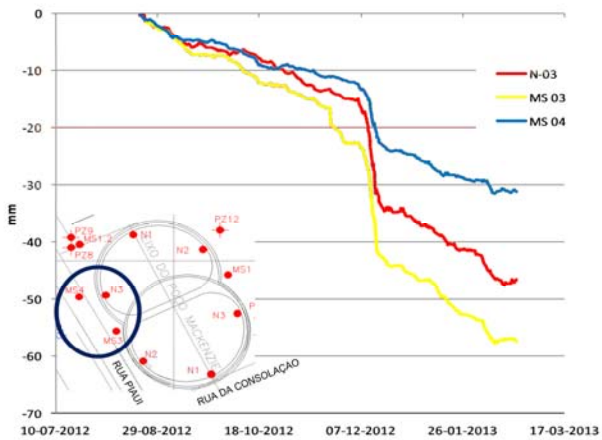


Figure 3. Evolution of the monitoring points N3 MS-3 and MS-4.

Figure 4 shows the evolution of the vertical movements in three points situated inside the shaft: surface (N1), at a depth of 7 m (N6) and at a depth of 21 m (N12)

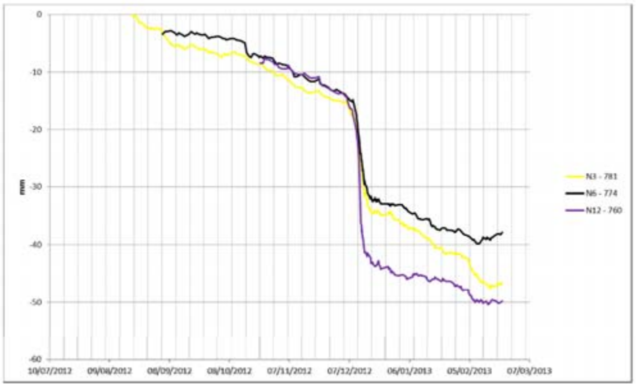


Figure 4. Evolution of the verticals movements of the reference points N1, N6 and N12.

From Figure 4 three important points are concluded:

I.- Until the soil piping in the bottom of the shaft, on December 8th 2012, the support of the shaft sank evenly.

II.- During the soil piping process point N12 sank up to 42 mm, while the points N1 and N6 sank up to 33 mm.

III.- After the soil piping three reference points have continued sinking, with a similar rate as before the fluidization.

3 BACK ANALISYS

In order to specify the conditions of the soil piping occurred in the shaft, on December 8th 2012, a "back analysis" of which aim was to reproduce the vertical movements of the monitoring points MS-3 and MS-4, represented in figure 3, was done with the following construction process:

I.- Mackenzie I Shaft Construction.

II.- Mackenzie II Shaft construction, excavating up the level 750 m, in which the soil piping occurred.

The following sections present the back analysis performed.

3.1 *Stratigraphy*

The ground on which the excavation Mackenzie Shaft was carried out is constituted by an alternation of clays and sands, until the bottom of the excavation where the ground is a sandy silt, as shown in Figure 5

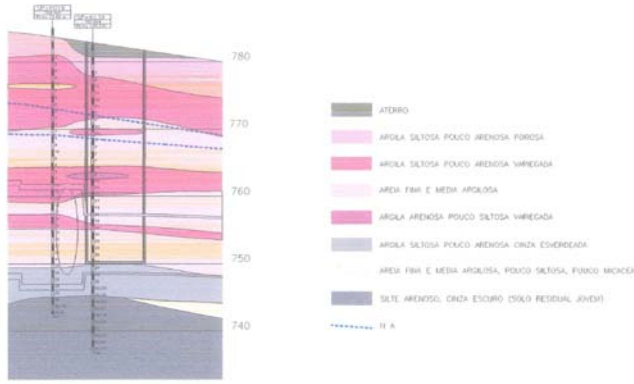


Figure 5. Shaft Mackenzie Stratigraphy.

The beginning of the Mackenzie Shaft is at level 781 and the bottom of the excavation at level 749, while the original water table varies between the levels 766 and 768.

The grounds properties which were considered are shown in Table 1.

Table 1.Ground properties.

Layer	Description	γ (kN/m³)	c' (kPa)	ϕ (°)	ψ (°)	K_0
1	Anthropic fill	16	10	20	0	0,83
3AgP1	Porous silty clay	16	20	27	-10	0,60
3Ag1	Silty clay	19	40	24	0	0,90
3Ar1	Clayely sand	19	3	30	0	0,80
3Ag1	Silty clay	19	40	24	0	0,90
3Ar1	Clayely sand	19	3	32	0	0,80
4Ag1	Silty clay	21,5	100	21	0	0,80
5SR	Sandy Silt	21,5	35	30	0	0,80

To characterize the terrain deformation process the theory of "large strains" was adopted, which assigns to the ground decreasing modules when the deformation increases, Hardin-Drnevich (1972).

The Cam-Clay was partially used as a constitutive model, since it takes into account that the deformation modulus decreases when the deformation rises.

3.2 *Geomechanical Model*

To carry out the "back analysis" a 3D model was performed, as shown in Figure 6, which has a length and width of 170m and 60m height.

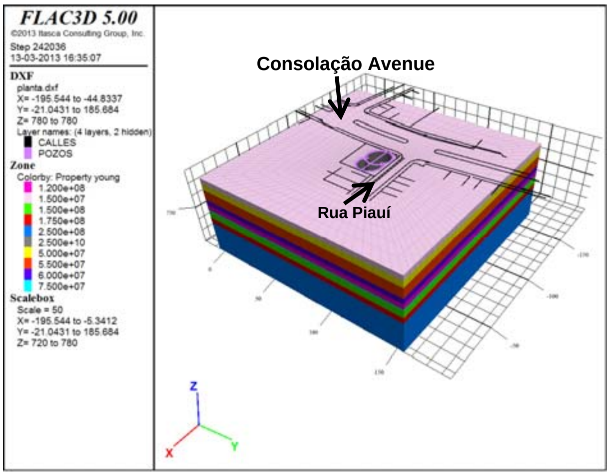


Figure 6. Geomechanical Model.

In this model, the Z axis corresponds to the direction of gravity, the x-axis to the direction of Piauí Street and the Y axis to the direction of the Consolação Avenue.

Figure 7 shows the distribution the water table in the model.

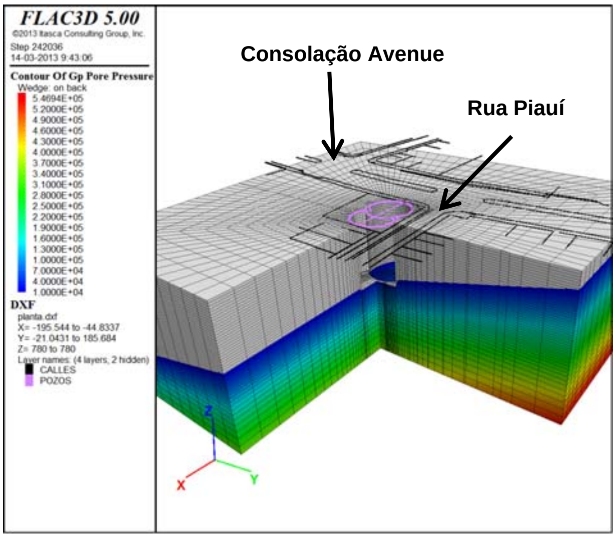


Figure 7. Position of the water table in the model.

One of the most important aspects of this model is the simulation of the junction between the primary lining Mackenzie I and II Shafts, for which we considered the four interfaces shown in Figure 8.

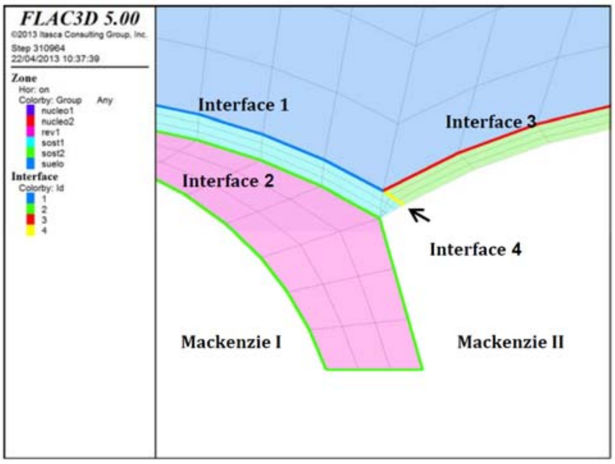


Figure 8. Interfaces to simulate contact between the linings of the shafts.

These four interfaces control numerically the following connections:

Interface 1. Contact between the Primary lining and the ground in the Mackenzie I shaft. The strength of this interface has been characterized by 2/3 of the shear strength of the ground with which it is in contact (reduction of cohesion and friction angle).

Interface 2. Contact between the Primary and the Secondary Lining of Mackenzie I Shaft, materialized through a waterproofing layer, characterized by $\phi = 20$ and $C = 0$.

Interface 3. Contact between the Primary lining and the ground in the Mackenzie II Shaft. This interface has been characterized as Interface 1.

Interface 4. Contact between the Primary lining Mackenzie I shaft and Mackenzie II shaft. This interface is characterized by a null cohesion and a friction of 32°.

3.3 *Vertical displacements settings*

The first Back Analysis calculations started simulating the Mackenzie I shaft excavation and continued with the sequential excavation of the Mackenzie II shaft up to the 750 level, which was the position of the excavation at the bottom of the shaft, where the soil piping occurred on December 8th 2012.

Figure 9 shows the distribution of the subsidence on the surface of the model, once it has reached the equilibrium after excavating the Mackenzie II shaft up to 750 level.

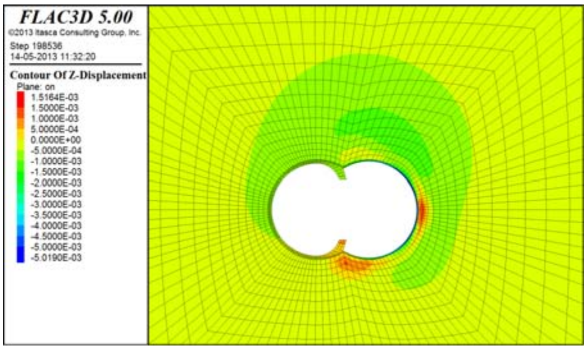


Figure 9. Subsidence at the surface, if the interface 4 works due to friction along the entire length of the shaft.

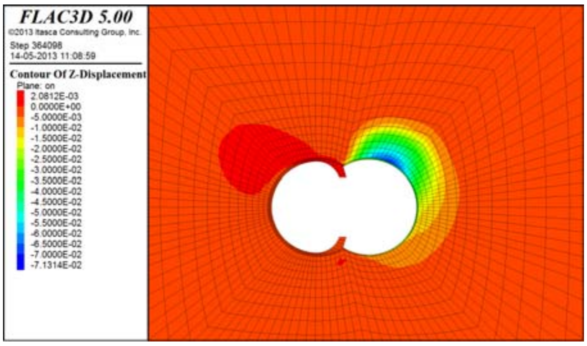


Figure 10. Distribution of subsidence if the junction between the primary linings of the two shafts works properly at a depth between 26 and 32 m.

This distribution of movements was obtained assuming the hypothesis that the junction between the primary linings of the Mackenzie I and II shafts worked properly along the entire length of the shaft.

Based on the information contained in Figure 9, the following conclusions were obtained:

I.- The maximum subsidence calculated is between 1 and 5 mm; values below 60 mm that were measured at the point MS-3.

II.- The space distribution of the subsidence does not match the actual measurements obtained during the construction, since the calculated maximum movements are on both sides of the axis of the two shafts and, in the construction they are concentrated closer to Rua Piauí.

These strong divergences between the calculations and the real movements indicated that the hypothesis that the Interface 4 worked properly across the shaft's length is incorrect.

Therefore new cases were calculated assuming that the length in which the Interface 4 worked properly was decreasing and it was found that when the Interface 4 was working in depths between 26 m and 32 m is when the results of the calculations fit in better with the real situation, as shown in Figure 10.

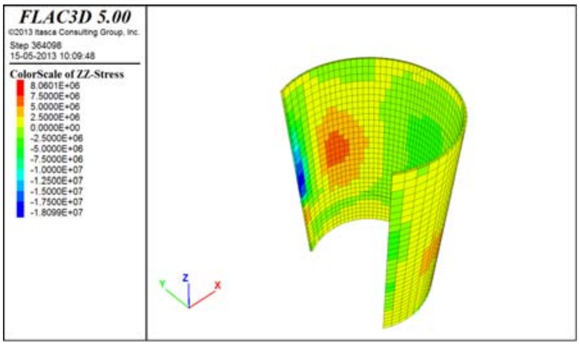
This figure shows that the maximum verticals movements of the ground are concentrated between the Mackenzie shaft and the Piauí Street, just as it happened in reality.

Moreover, the vertical movements calculated for the position of the monitoring points MS-3 and MS-4 are between 60 and 70 mm, range in which are included the measurements made in points MS-3 and MS-4.

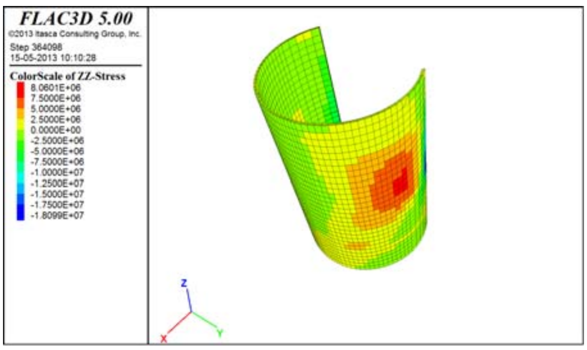
3.4 *Induced stresses on support shaft*

After adjusting the 3D model to the real conditions that occurred during the construction of the Mackenzie Shaft it was calculated the stress induced in the shaft support due to downward movements which affected the construction.

From the calculations done with the assumption that the joint between the two primary supports worked partially, the distributions of the stress induced in the two faces of the support have been obtained, as shown in Figure 11.



I. Inner side of the support.



II. External surface of the support.

Figure 11. Stress distribution of σ_Z in the shaft support structure (Positive values are tractions).

This figure shows that in the support structure has created a tensile area, with a maximum value of about 8MPa, which is a high value and probably would have produced a horizontal tensile crack.

3.5 *Factor of safety before the soil piping*

Calculations done have established that the safety factor (SF) when the excavation reached the 750 level was 1.1, which justifies the occurrence of the soil piping of December 8th 2012. The SF has been determined according to the criteria of Duncan (1996).

Figure 12 shows the elements of the model around the shaft in which SF=1.1, which would be the ones to be mobilized during the soil piping. It can be seen that most of these elements are located between the Mackenzie Shaft and Piauí Street.

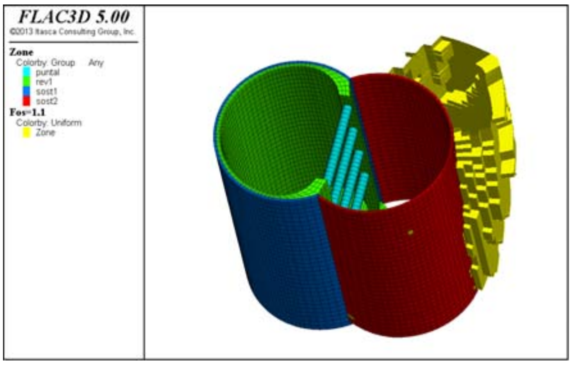


Figure 12. Elements with SF = 1.1 to reach the excavation at a level of 750.

4 SOLUTION ADOPTED

The solution adopted to complete the construction of the Mackenzie Shaft included two distinct actions:

I.- Reinforcement of the primary lining of the shaft with three rings of shotcrete of 3 m high, to prevent the effect of induced tensiles of the produced movements.

II.- Construction of a conical wall of jet-grouting piles to prevent the movement of the shaft's structure and isolate the bottom of the the excavation from the effect of water pressure.

Picture 3 shows a view of the execution of one of the reinforcement rings and Picture 4 shows the reinforcement ring at the bottom of the excavation.

Picture 3. Reinforcement of the primary linings between levels 780 and 777.

Picture 4. Reinforcement of the primary linings between levels 755 and 758.

The construction of a conical wall of jet grouting piles was designed, drilled from inside of the shaft to the level 755, as shown in Figure 13.

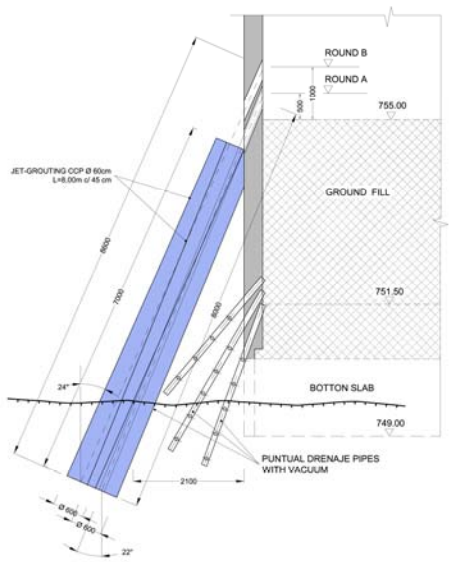


Figure 13. Wall of jet-grouting piles.

The jet-grouting piles are around the entire of Mackenzie II shaft, although in the nearness of Piauí Street two rows of piles were established, as shown in Figure 14.

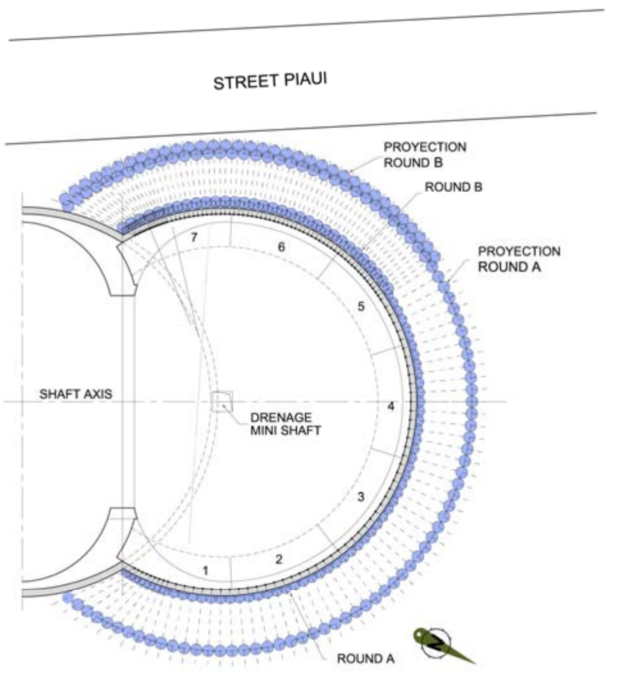


Figure 14. Disposition in plan view of the jet-grouting piles.

5 VERIFICATION OF THE PROPOSED SOLUTION

To verify the proposed solution the 3D model employed in the back analysis was modified by adding jet-grouting piles, as shown in Figures 15 and 16.

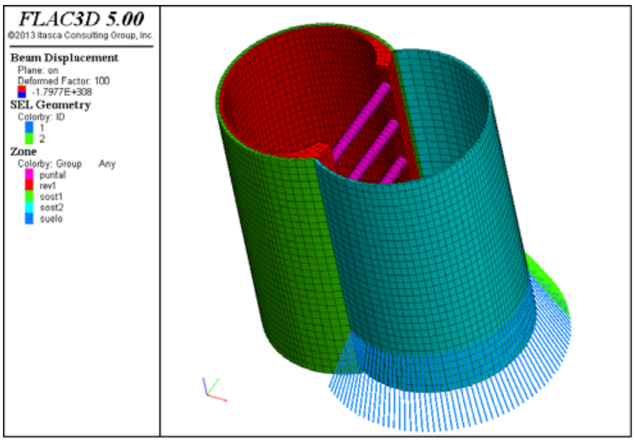


Figure 15. 3D model with jet-grouting piles.

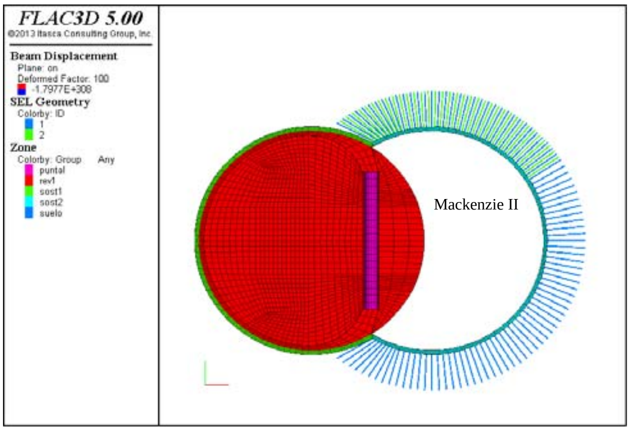


Figure 16. Modeling in plan view of the jet-grouting.

This model was solved with the software FLAC 3D simulating the stacked lands removal in the shaft, between levels 755 and 750 and the excavation of the meter of ground missing to reach the bottom of excavation, at level 749.

Figure 17 shows the deformation of the jet-grouting piles at the end of the excavation and also represents the grounds having a SF = 1.4, which is the same calculated for this solution, which was considered satisfactory.

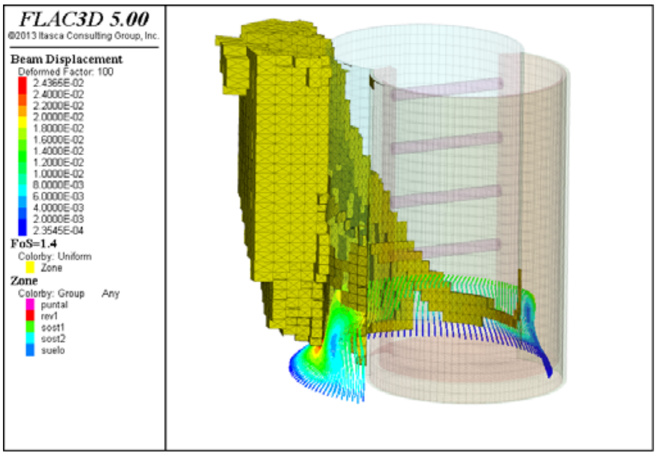


Figure 17. Deformation of the jet-grouting piles and grounds with SF = 1.4.

6 ENDING OF THE CONSTRUCTION

The construction of jet-grouting piles was carried out without incidents since the end of June 2013 when it was proceeded to remove the stacked ground in the bottom of the shaft; between levels 755 and 750

Picture 5 shows a view of the removal process of ground and Picture 6 shows the bottom of the shaft completely clean, June 20th 2013.

Picture 5. Removal of ground at the bottom of the shaft.

Picture 6. Bottom of the shaft 20/06/2012 at level 750.

After completing the removal of ground it was proceeded to continue the excavation, for which the bottom of the shaft was divided into 7 sectors, as shown in Figure 14.

During the excavation of sectors 2, 5 and 7 water inflows occurred, with a caudal close to 5 l/s. Although this rate was not very important, the clayey nature of the ground towards the bottom of the shaft was full of mud. This problem was controlled with a system with vacuum drainage pipes and a permanent drainage at the bottom.

Picture 7 shows a snapshot of the shotcrete after ending the excavation in Sector 5, which was one of the most controversial.

Picture 7. Shotcrete projected in Sector 5.

Finally, the excavation and execution of the primary lining in Mackenzie II shaft finalized on July 24 th, 2013.

Picture 8 shows a view of the bottom of the shaft during the phase of waterproofing, after the construction of the slab concrete botton.

Picture 8. Waterproofing Mackenzie shaft, after finishing the primary lining.

7 CONCLUSION

The construction of the second shaft of the Mackenzie Access of Higienópolis Station of Sao Paulo Line 4 subway had to overcome a serious problem occurred on December 8th 2012; when the bottom of the shaft was found a soil piping, reaching 31 m excavation depth.

To solve this problem a detailed analysis of monitoring data was performed, which allowed to establish a baseline to perform a back analysis which allowed specify the causes of the soil piping and the movement effects on the primary support of the shaft.

Once the phenomenon affecting the Mackenzie II Shaft was well known, a solution was proposed to end the excavation based on jet-grouting piles; which were executed from inside of the shaft.

This solution has been very effective, since it allowed the ending of the excavation on July 24th, 2013.

The main lessons learned from this experience to be applied in future cases are the following ones:

- It is absolutely necessary to have the most accurate knowledge of the terrain characterization in the exact place in which the works are going to be executed.
- The calculus performed during the design phase must be sufficiently representative and accurate in order to use them during the works.
- During the execution of the works it should be checked that the hypothesis predicted during the design phase are being fulfilled.

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