# Porto Maravilha project – infrastructure works.

M. Waimberg and F. R. Stucchi
*EGT Engenharia, São Paulo, Brazil.*

F. P. de Barros
*Engecorps Engenharia, São Paulo, Brazil.*

O. Marchetti
*Estra Engenharia, São Paulo, Brazil*

W. Bilfinger
*Vecttor Projetos, São Paulo, Brazil.*

J. Takahashi
*Petrometal, São Paulo, Brazil.*

ABSTRACT: The Porto Maravilha Operation, in Rio de Janeiro, comprises a series of infrastructure works, among them a new road system in the old port area, namely the Binária and Expressa ways. The Saúde tunnel is part of the Binária way, and was constructed in the Saúde hill, part of a deactivated old quarry, having two three-lane carriageways plus a parallel LRV track. With a 300m$^2$ cross section, the excavation was performed in three steps – two lateral galleries and a central one – through gneiss biotite rock mass and saprolite. Due to asymmetrical plan geometry, asymmetrical lateral confinement and small rock cover a structural reinforced concrete lining was adopted. This lining, up to 1m thick, is subjected to major internal forces due to asymmetrical overburden. At last, settlement monitoring results are presented.

## 1 INTRODUCTION

The Porto Maravilha Urban Operation consists of reurbanization and readaptation of a 5-million-square-meter area in the port region of Rio de Janeiro, aimed at requalification of ground use and its real estate and socioeconomic development.

The intervention consists of a series of urban infrastructure works, equipment, street facilities and landscaping. Among the infrastructure works, a new road system is one of the main features, with the demolition of the Perimetral viaduct and construction of underground roads increasing the traffic capacity of vehicles passing through the region and its internal flow. These works are called structuring works, composed of the so called *Binária* and *Expressa ways* (see Figure 1), and they are built by the Porto Rio Consortium, formed by OAS, CNO and Carioca Engenharia companies, and designed by a consortium formed by EGT Engenharia and Engecorps companies.

The *Binária way* is formed by a surface road along Venezuela street and by an underground road from 1º de Março street, next to São Bento hill, to the surroundings of Saúde hill.

Both roads have 3 lanes each for vehicular traffic and shoulder lanes for emergency, totaling 11.80m of horizontal clearance.

The underground road is composed by around 450m of trenches and around 1km of tunnels excavated using conventional tunneling technique, mostly in rock.

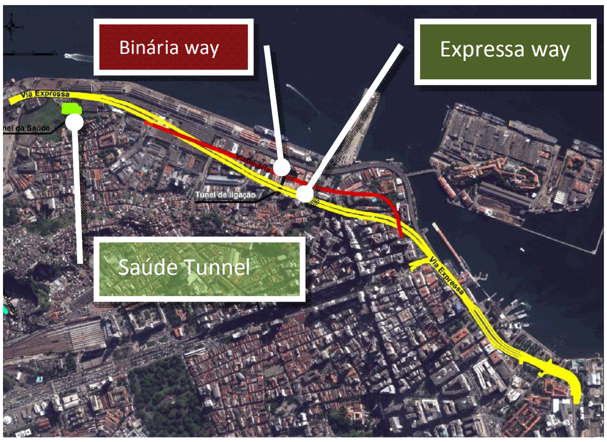


Figure 1. General plan.

The tunnel under the Saúde hill, excavated above surrounding ground surface level, comprises both roads of the Binária way, in addition to a LRV (light rail vehicle) track to be implemented in the region.

With an excavation section of almost 300m², it goes through what is left from an old quarry, in mixed rock/saprolite section, under a residential condominium on the top of the hill.

The hill is highly asymmetrical, with an almost-triangular plan format, so that the tunnel has very skew portals, up to 45º (see Figures 2 and 3) and a very asymmetrical loading. In addition, on the ocean side, the lateral confinement is minimum.

These characteristics conditioned the adopted structural and construction method. The tunnel lining consists on a reinforced concrete structure that stands for applied loads subjected to major internal forces. There is practically no arching of vertical loads, since there is an overburden of approximately half tunnel diameter.

The excavation is divided in three parts, two side tunnels and a center gallery, with the ocean side tunnel providing adequate support for the asymmetric load applied to the structure.

Right now, the Saúde tunnel is concluded and the others are in progress. In this article, the characteristics of this project will be presented in details, including applied treatments, lining sizing and performance throughout the excavation.

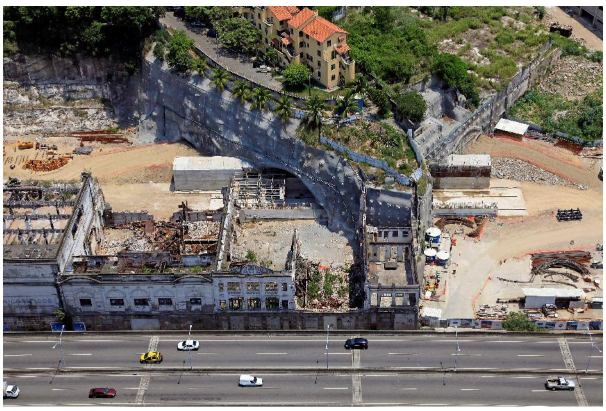

Figure 2. Construction works in a top view from the ocean side

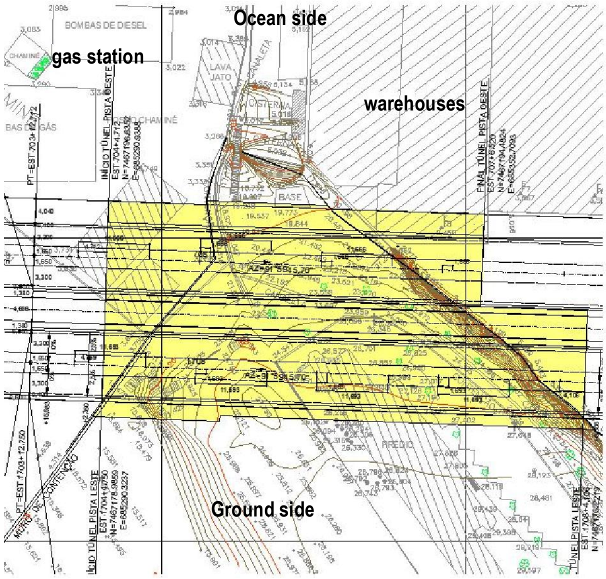


Figure 3. Tunnel plan.

## 2 LOCAL GEOLOGY

The Saúde hill is constituted by a migmatized-gneiss biotite rock mass, with pegmatite intrusions, overlaid by granite gneisses, topographically increasing behind Cibrazem's warehouses, between the port area and the Central do Brasil railway network.

This hill is narrow and smaller than 100m in NE-SW length, located within the urbanized area. It is situated between elevations of 15 to 30m, with two outcrop faces, like sheer rock faces: the first, in rock, at the back of the warehouses and the other one, in saprolite, next to the former Chaminé gas station.

The landfill thickness verified by boreholes is even surprisingly thick in elevated areas of the hill, as well as the thickness of the gneiss saprolite. The rock mass, made of gneiss biotite, has average geomechanical properties, with predominance of class III mass, according to Bieniawski.

Initial geological studies were developed with the performance of 7 boreholes.

During the Basic Project studies and for the Executive Project, 11 additional boreholes were performed, at a total of 267 m drilled length. From these, 4 holes were vertical, 2 horizontal and 5 with 30° to 45° inclination in relation to the vertical axis.

This borehole arrangement was due to the difficulty of placing the sounders in the steep topography and the almost complete occupation of the area by houses.

Figures 4 and 5 below indicate the longitudinal sections along the ground side gallery and the central gallery, for the LRV, respectively.

The groundwater level is immediately below the tunnels floor, so as not to interfere in its excavation.

Saprolite, as the transition material between soil and rock is named in the project, is excavated mechanically with the help of a pneumatic hammer and can be characterized as a transition material between soil (young residual soil, saprolitic soil) and the rock mass situated below the so-called rockhead.

This horizon is, by definition, heterogeneous in its composition. It comprises the material which can no longer be classified as soil and cannot have its properties analyzed by traditional methods of soil mechanics, or analyzed by concepts of rock mechanics, which considers the interaction of rock blocks with a certain fracturing pattern.

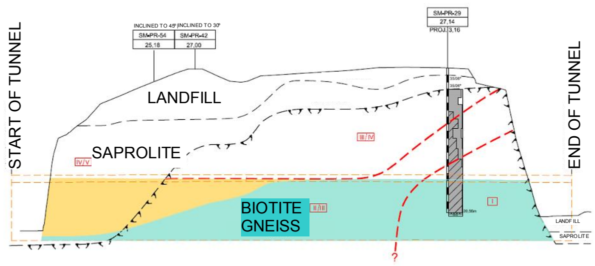


Figure 4. Profile of the ground side gallery

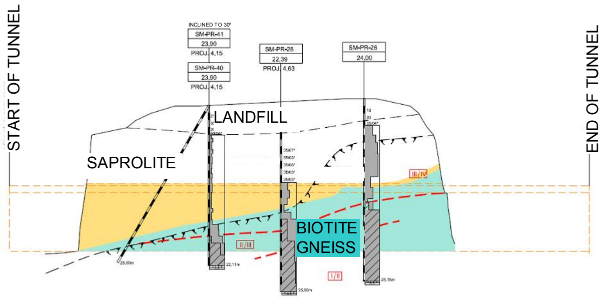


Figure 5. Profile of the central gallery

## 3 STRUCTURAL SOLUTION

### 3.1 *Geometry*

The almost 300m² cross section of the tunnel was divided into three galleries - East and West roads and, in the middle, the LRV, with total excavation of 35m width x 10.5m height (see Figure 6).

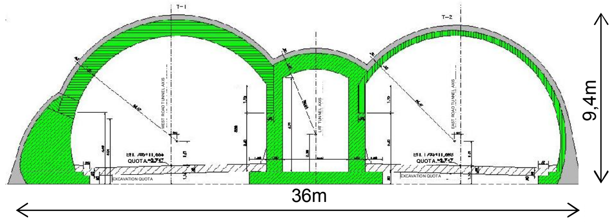


Figure 6. Cross section.

In addition to the strong asymmetry due to the skew portals in plan view, as can be seen in Figure 2, there is also a global transversal asymmetry.

On the ground side, the excavation is performed almost entirely in the rock mass, with around 10m rock cover. However, on the ocean side, the rock cover is very small and the saprolite invades the cross section in most of the excavation (see Figure 7).

On the ocean side, lateral confinement is very small, because the excavation is carried out practically next to the external face of the rock mass. In addition, the rock mass edge on this side has approximately 1/4 of the length of the ground side (see Figure 2).

The highly unbalanced loading between the ground side and ocean side is sustained by the concrete lining structure and supported by the tunnel foundation. Thus, due to the skewing of the portals, an extension of the tunnel structure was built on the portal at both ends.

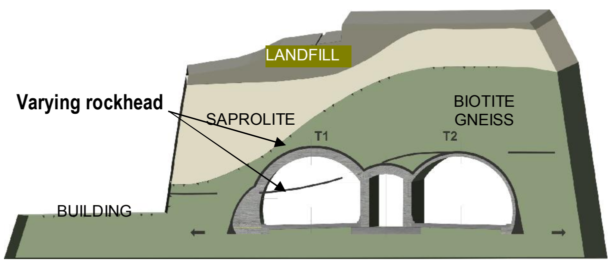


Figure 7. Geological cross section.

### 3.2 *Loadings*

The rock cover is small or inexistent. Thus, there is almost no arching of vertical loads and the tunnel lining was designed to support the entire vertical load applied (γ.h), an overburden of half tunnel diameter.

Due to the asymmetry in plan view, the total load applied to the ground side gallery is much higher than the one applied to the ocean side gallery. While on the latter one vertical load varies from 120 to 200kN/m$^2$, on the former vertical load varies from 240 to 300kN/m$^2$.

This unbalanced load "pushes" the structure toward the sea and cannot be balanced by the rock mass. The reinforced concrete structure is responsible for the equilibrium of the applied loads, resulting on important internal forces in the lining. To account for reaction on the ocean side wall, a large footing was provided after filling part of the excavated side drift with concrete secondary lining.

## 4 CONSTRUCTIVE SEQUENCE

Before the excavations, to provide confinement to the rock mass, tiebacks were installed on the shotcrete sprayed on the three exposed faces of the rock mass (see Figure 8). In part of the (less skew) east face, supplementary anchor bolts were installed to the already existing tiebacks.

Initially, the excavations begin by a side drift on the ocean side gallery. This side drift has a reinforced concrete lining (with lattice girder ribs), sufficiently rigid and with proper strength to act as support, like a footing to the whole tunnel gallery.

At the same time, a center tunnel is excavated for the LRV gallery, in a cross section divided in portions. As the excavation progresses, the secondary lining and two reinforced concrete walls are carried out. These walls act as support to the lining of the future ocean side and ground side galleries, respectively.

Likewise, for construction purposes, it was chosen to excavate a side drift on the ground gallery as well, parallel to the previous excavations.

After finishing the side drifts, the excavation of the ocean side gallery is started. As the excavation progresses, the shotcrete primary lining is applied, with lattice girders. Then the partial demolition of the primary lining of the pilot center tunnel is carried out and the reinforced concrete final lining is performed.

At last, the sequence is repeated for the ground side gallery.

Simultaneously to the performance of the secondary lining, the extended tunnels before the portals have their concrete done.

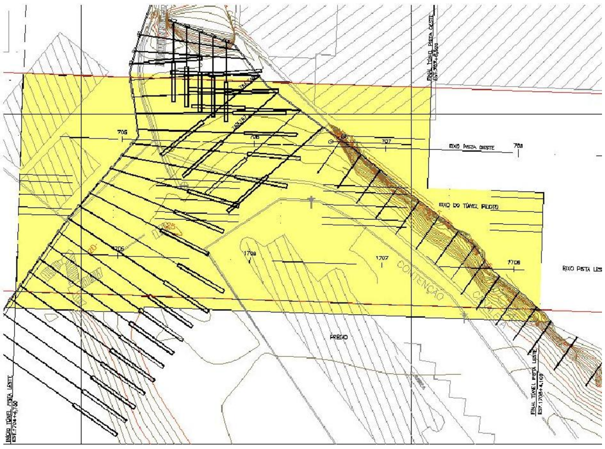

Figure 8. Rock mass confinement

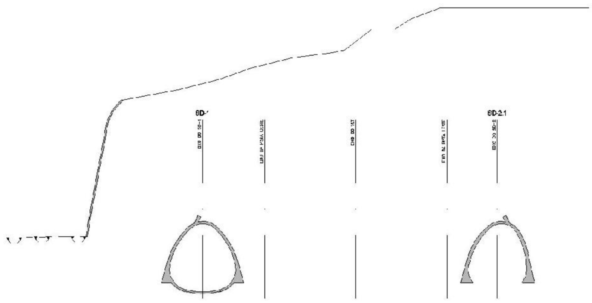

Figure 9a. Side drifts excavation

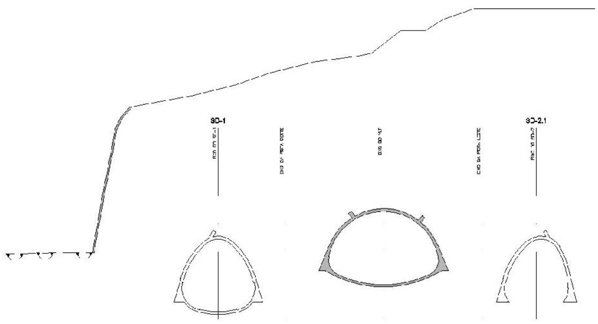

Figure 9b. Center tunnel excavation

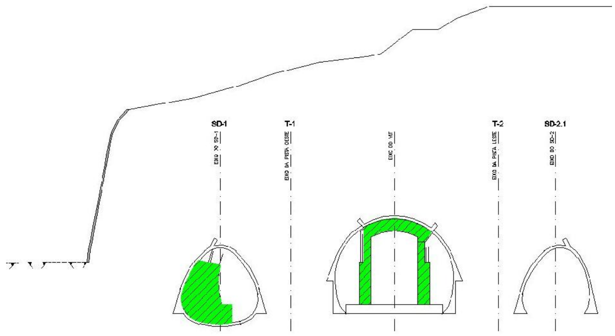

Figure 9c. Concreting of the internal structure of the center tunnel and side-drift of ocean side gallery

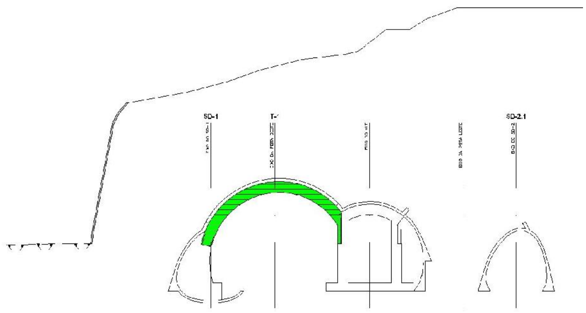

Figure 9d. Excavation and concreting of the ocean side gallery

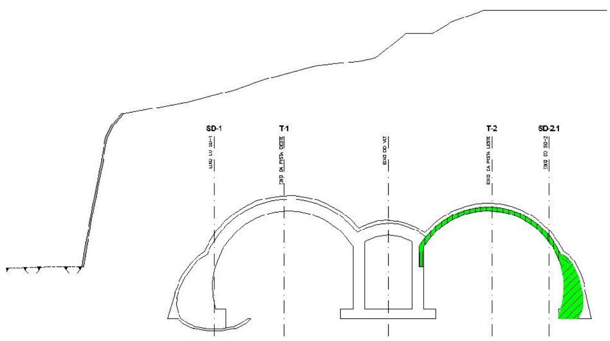

Figure 9e. Excavation and concreting of the ground side gallery

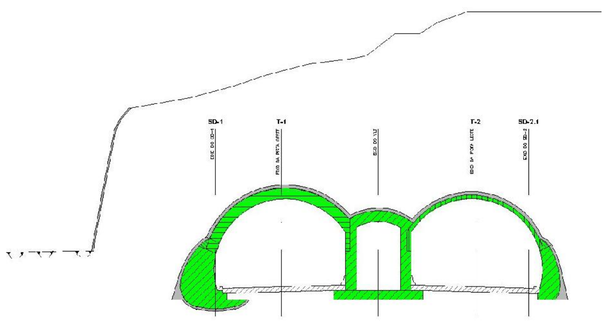

Figure 9f. Concrete lining after conclusion

Figure 9. Construction sequence

5 NUMERICAL RESULTS

At first, simple numerical models using bar elements were prepared to evaluate the internal forces acting on the concrete lining, in every step of the construction sequence, at different cross sections of the tunnel.

These forces lead to the consideration of a 30cm thick primary shotcrete lining all over the tunnel and a secondary cast in place concrete lining, 35cm thick at the ground side gallery, and 100cm thick at center and ocean side gallery, as well as at vertical walls in the center gallery. As mentioned before, these elements although subjected to smaller vertical loads are responsible for the global equilibrium of the tunnel.

Because of the previously mentioned asymmetry, significant longitudinal effects are expected. Thus, a complete finite element model of the final structure was prepared using SAP2000 software. In this model soil/rock mass is considered as a set of loads applied to the concrete lining. Lateral confinement is modeled by springs representing the ground stifness.

Some results are presented on figures 10a to 10d.

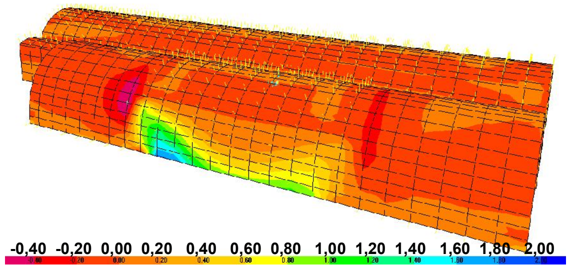

Figure 10a. Longitudinal bending moments

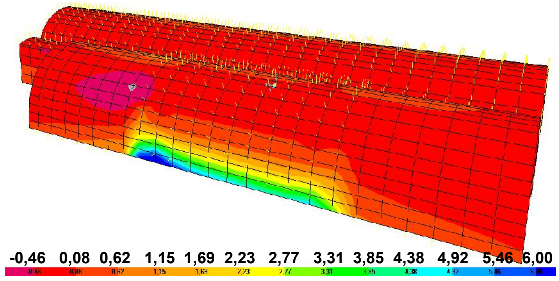

Figure 10b. Transverse bending moments

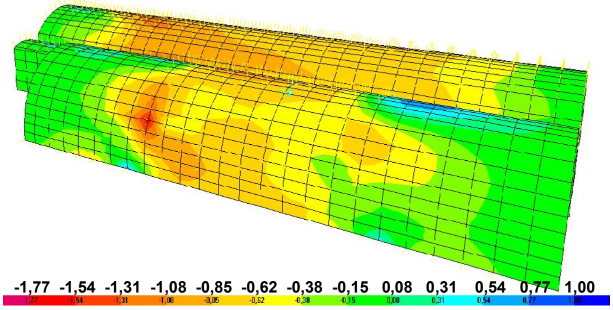


Figure 10c. Longitudinal forces

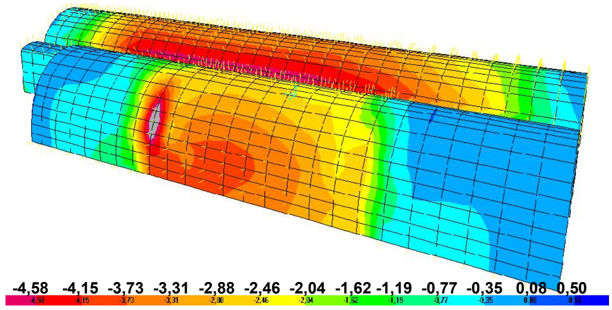


Figure 10d. Transverse forces

Figure 10. Internal forces on the final lining (MN, m)

Maximum bending moments achieve a 3.300kNm/m value on transverse direction, at the base of the structure where concrete lining is thicker. At the top, there is a 500kNm/m bending moment. On longitudinal direction bending moments of 500kNm/m and 100kNm/m act at these points, respectively.

Compressive forces of 4.000kN/m act on transverse direction. Due to longitudinal asymmetry, longitudinal compressive and tension forces around 600kN/m occur.

## 6 INSTRUMENTATION

For monitoring the development of the excavations, a set of instrumented sections was installed, with surface settlement plates, deep settlement gauges (see Figure 11), excavation inner sections for control of convergences and leveling pins on the surface buildings.

In general, the excavation of the rock and saprolite tunnels did not generate significant strains and displacements internally to the excavation, as expected.

However, buildings on the surface, built with masonry on shallow foundation, had minor to average damages, such as cracks and fissures on masonry walls, caused by surface settlement. This settlement was caused partly due to the deformations caused by excavation, but mostly due to the vibration caused by rock blasting.

The following are the main results of this instrumentation, with data acquired by Bureau de Projetos e Consultoria Ltda. It is found that both deep and surface settlement went beyond, in some cases, the expected values, indicated by the green line on the charts (Figures 12 to 15). These values were around 15mm for settlement near the excavation and around 10mm on the surface. The red line corresponds to the warning values established for the control of the technical supervision group.

Settlements higher than the expected are likely due to the high variability found in the rock mass and to the differences between the geological profile provided in the project and the one actually found in the excavation.

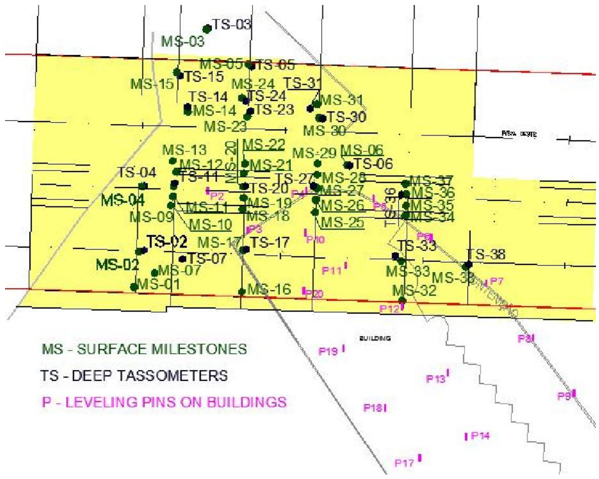


Figure 11. Monitoring plan

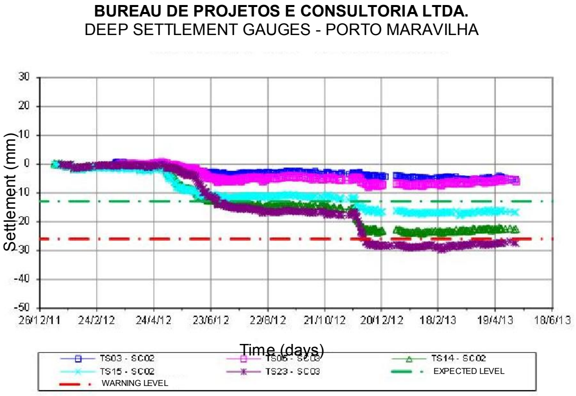


Figure 12. Deep settlements – gauges 3, 5-1, 14, 15 and 23

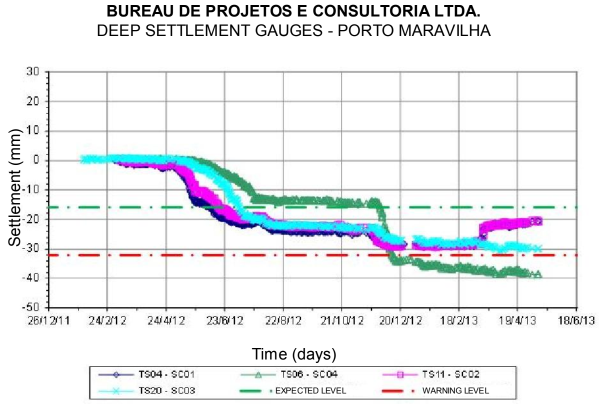


Figure 13. Deep settlements - gauges 4, 5-2, 11, 20

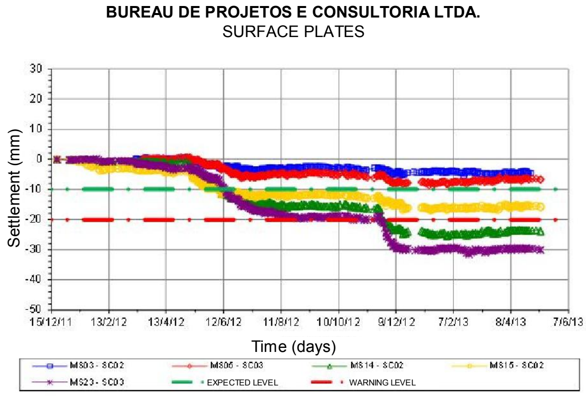


Figure 14. Surface settlements – plates 3, 5, 14, 15, 23

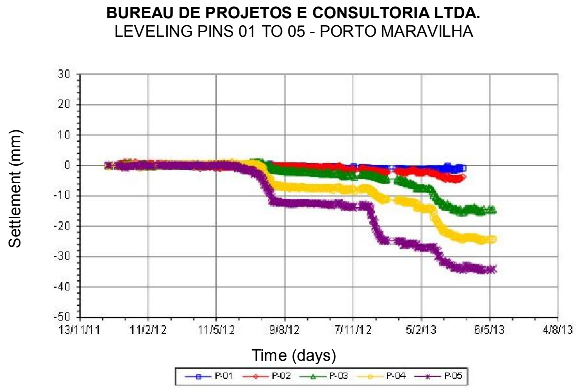


Figure 15. Settlement on buildings – pins 1 to 5.

## 7 CONCLUSION

In this paper we presented some difficulties involved in the construction of the Saúde tunnel, in Rio de Janeiro, either due to the geotechnical environment and geometrical constraints.

Thanks to a construction sequence which partializes the excavation, to the applied treatment and to a structural concrete lining the construction was very successful, with little influence on the surrounding area and the tunnel is now open to vehicles traffic (sea Figure 16).


Figure 16. The tunnel concluded

## ACKNOWLEDGEMENTS

The authors thank the companies from the Project Consortium and the Construction Consortium of the construction works of Porto Maravilha for making available all data presented herein.