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PAT TBM improving—a case of study to Metro São Paulo

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ABSTRACT: In the final stage of Line's 5 tunnel construction—São Paulo Metro (Chácara Kablin Station—VSE Dionisio da Costa), by the use of TBM-EPB, a PAT (Plan for Advance of Tunnel) instrument has been generated for improving the previous referenced TBM face pressure, principally due to variation of the underground water level and to the new up-pressure contribution related to the load of a new building edified above the tunnel route, of 48 m height and with 2 underground levels of 7 m depth. The up-pressure contribution of the new building, in the face TBM pressure calculation, has been considered as part of the footing pressure, evaluate by model based on the theory of elasticity. The result of the TBM's parameters recorded during the excavation, the monitoring data and the absence of building damage shows the integrity of the methodology used.

1 INTRODUCTION

1.1 *Background to the problem*

In the final stage of Line's 5 tunnel construction—São Paulo Metro (Chácara Kablin Station—VSE Dionisio da Costa), by the use of TBM-EPB, the update of the referring earth pressure proved to be necessary, due to different underground water levels foreseen at the final design stage and due to a new unconsidered building's construction above the tunnel route (Figure 1). The building (Commercial—Offices Francisco de Vitoria Street), of 48 m height and with 2 underground levels of 7 m depth, is founded on footings of different area, where the generated distribution of the vertical stress ("pressure bulb") overlap with the excavation tunnel section within Pk 21+196 and Pk 21+220. The update of the referring TBM-EPB earth pressure was made by the Metro supervision team, following the general concept of PAT (Plan for Advance of Tunnel) conceived also as a *dynamic tool for interrelating design and construction of an underground work* (Guglielminetti et al. 2007), in order to optimize the excavation methods and/or the excavation parameters quickly and therefore to meet the excavation plan's needs, minimizing the settlement risk and structural damage risk.

1.2 *Referring characteristics*

The geological setting of the considered excavation sections (Pk 21 + 196 – Pk 21 + 220) involves the

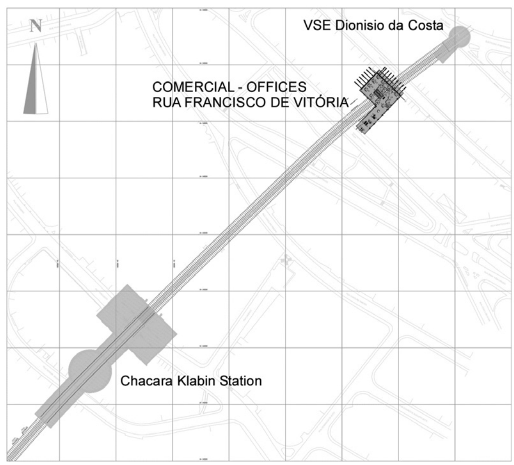

Figure 1. Localization of the Commercial-Office in Francisco de Vitoria Street.

soil of the Neogene Formations of São Paulo overlying (clay with thin layers of fine-medium sand) and Resende underlying (stiff clay and clayey sand). The building is founded on clay of the São Paulo Formation. The underground water level measured by the piezometers at April–May 2016 is 3–5 m down respect to the one foreseen in the final design (Figure 2).

The geotechnical parameters used to evaluate the earth pressure in this section and the expected deformations are showed in Table 1 and Figure 3.

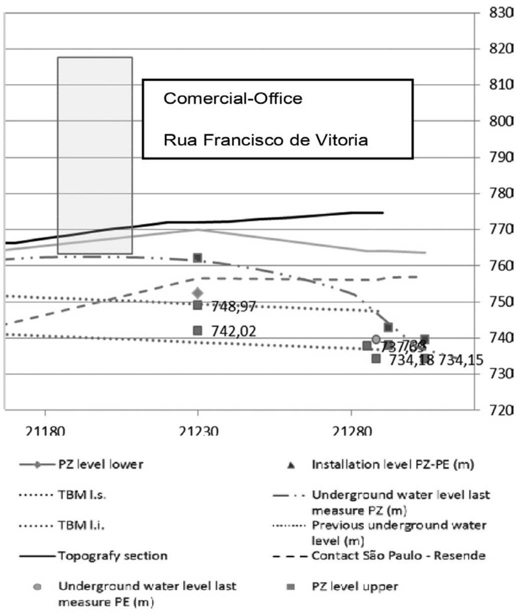

Figure 2. Geological-hydrogeological sketch in relation to TBM tunnel and building. The piezometers measures points are referred to April–May 2016. (SP—São Paulo Formation. RE—Resende Formation).

Table 1. Main geotechnical characteristics of the soils, involved in the considered section.

Geological formations	Geotechnical units	c′ (kPa)	ϕ' (°)	γ (kN/m³)	Es (MPa)	K (cm/s)
São Paulo	Fine Medium Sand	10	32	19,00	40	5,0E-04
	Clay	40	20	18,00	20	5,0E-06
Resende	Stiff Clay	60	20	20,00	70	5,0E-07
	Stiff Clayey Sand	60	22	20,00	70	5,0E-07

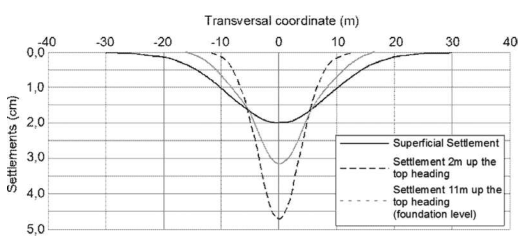

Figure 3. Greenfield settlement at Pk 21+200. VL = 0,5%, K = 0,4.

Figure 4. Sketch of the monitoring system carried out for the Commercial-Office in Francisco de Vitoria Street.

The greenfield settlement induced by the tunnel excavation has been defined by the analytical method of New and O'Reilly (1991) considering the volume loss $V_L = 0{,}5\%$, the trough width parameter constant K = 0,4 (clay excavation), on the tunnel axis, the predicting magnitude of the settlement, resulting of 20 mm, superficial, 47 mm at 2 m up the top heading and 32 mm at 11 m up the top heading, at the foundation level (Figure 3). The building risk assessment for this structure was not defined in the final design and the same, for absence of the building details in the design stage, leaving liability to the Metro supervision team after its construction and during the excavation. The Metro supervision team has considered the risk as potentially expected damages.

The monitoring system has been done by the installation of superficial topographic marks, tassometers, inclinometers and optical target installed on the lower part of the building structural pillars, for settlement measurements and rotation evaluations (Figure 4), as well as piezometers for evaluations of level of the underground water. The frequency of the instrumentations readings is found to be variable between 2 and 8 measurements per day in relation to the position of the TBM.

1.3 *Evaluation of the face TBM pressure*

In the final design for the estimation of the face pressure, it was decided to use, among other analytical method mentioned by Guglielminetti et al. (2007), the method of Caquot-Karisel (1956) based on the plasticity theorems, integrated by Carranza-Torres (Guglielminetti et al. 2007, Napa-García et al. 2014), that permits the estimation of the internal pressure p_s necessary to maintain the opening stability at ultimate limit state, consider-

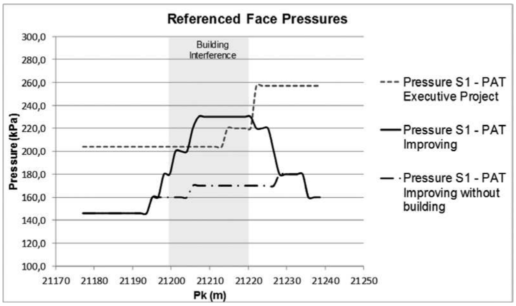

Figure 5. Referenced Face Pressure in the final design PAT, in the improving PAT considering the up-load of the building, in the improving PAT without the up-load of the building. (S1 = pressure sensor of reference).

ing a global factor of safety FS included into the formulation and different hydraulic conditions (Napa-García et al. 2014). In this case the hydrostatic pressure, a surcharge (20 kPa) and a safety factor FS = 2 has been considered.

The values of the referenced pressure are shown in the Figure 5. The variation on the underground water level and the new upgrading of the external condition, due to the construction of the building upper the TBM tunnel route, has induced the modification of the referenced pressure. The face TBM pressure evaluation was considering only the variation of the underground water level, due to the last piezometric registrations, and more important the external surcharge due to the completely load of the building are not considered a correct operating procedure, due to very high pressure values because it may produce an uplift of the ground and thus create a further differential deformation of the structure. Consequently for this particular case, it has been considered only the vertical pressure generated by a part of building load, added to the face TBM pressure evaluated by using the above mentioned method of Caquot-Kerisel (1956) integrated by Carranza-Torres (Guglielmetti et al., 2007). Therefore the face TBM pressure calculated in correspondence to the building was 230 kPa compared to 170 kPa without footing pressure contribution.

1.4 *Building foundation vertical stress contribution*

The distribution of the vertical stress in soil mass due to footing pressure has been defined by several methods. For the construction stage, for an easy and quickly reply, the Steinbrenner Chart (E.D. Chen and J.Y. Richard Liew. 2003), on the basis of the theory of elasticity, has been used to define the vertical stress at the tunnel level, in relation also to the rectangular footing geometry and to the semi-homogeneous soil mass above the tunnel, composed mainly by clay-stiff clay. Considering the principal sizes of the footing of 32 m^2, with a load of 14,400 kN applied, an up-pressure contribution of 60 kPa has been considered during the excavation down the central part of the building.

1.5 *PAT improving*

The improvement of the PAT, incorporating the redistribution of the previous referenced face pressure has been considered by taking into account the last survey campaign to measure the piezometric levels as well as the contribution of the up-load of the building. Moreover, to achieve this improvement, it has been considered also a pondering on the deformation behavior of the ground mass, by considering variation of the heading, lateral geological structuring, presence of others interferences in the area and the application procedure for the grout injection to fill the voids between the extrados of the lining and the excavated tunnel profile, at the tail shield.

The backfill grouting procedure and, in particular, the pressure and volume of the grout injected are parameters that influence, in secondary manner, the surface settlements as well as ground mass deformations. Once it is defined the theoretical volume gap for the backfill grouting, the pressure is related directly to the face pressure, even if it is a more difficult procedure to adjust during excavation.

Based on the analyses previously indicated, it was chosen to proceed with a gradual increase of the face pressure between the previous sections, characterized by lower pressure (160 kPa) and the building footprint (Pk 21 + 196 and Pk 21 + 220), considering an offset of about 10 m (7 rings—Figure 5).

2 RESULTS

During the excavation of the TBM the face pressures were maintained within the values indicated in the improving PAT (Figure 6), whereas the pressures for the backfill grouting have undergone changes respect the theoretical values due to operational choices.

The weight ground's mass value extracted from the screw conveyor of the TBM were maintained respect to the theoretical value as shown the ratio R in the Figure 7.

Regarding the TBM penetration rate, it can be seen that this parameter show constant values in the considered section (Figure 8).

Finally, the deformation of the soil mass during the TBM excavation has been mild and negligible

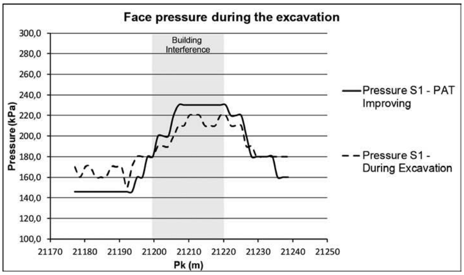

Figure 6. Referenced Face Pressure in the improving PAT considering the up-load of the building and the Face Pressure recorded during the excavation. (S1 = pressure sensor of reference).

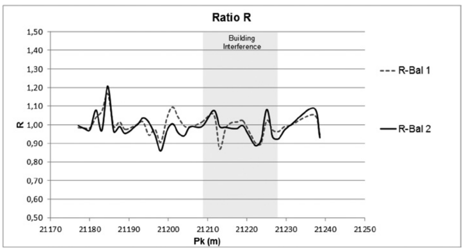

Figure 7. Ratio R, between the mass of the actually extracted material from the screw conveyor and the theoretical mass.

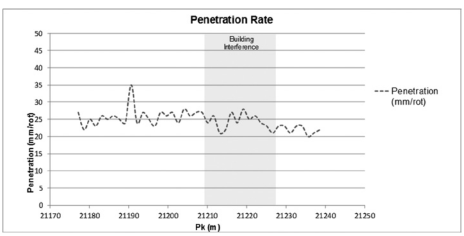

Figure 8. TBM penetration rate.

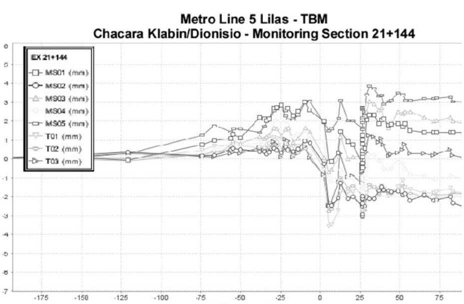

Figure 9. Monitoring Section 21+144 (by SACI-METRO system).

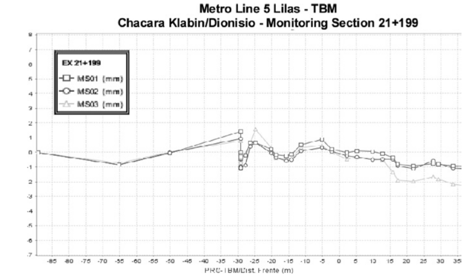

Figure 10. Monitoring Section 21+199 (by SACI-METRO system).

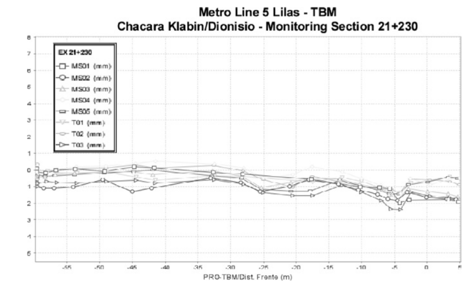

Figure 11. Monitoring Section 21+230 (by SACI-METRO system).

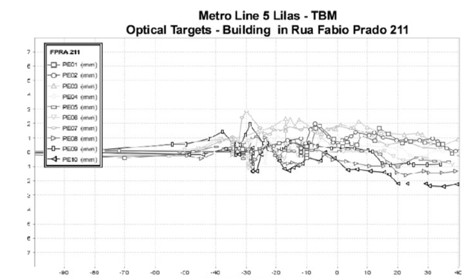

Figure 12. Monitoring lower part of the structural pillars of the building with optical targets. PE01 – PE10 (by SACI-METRO system).

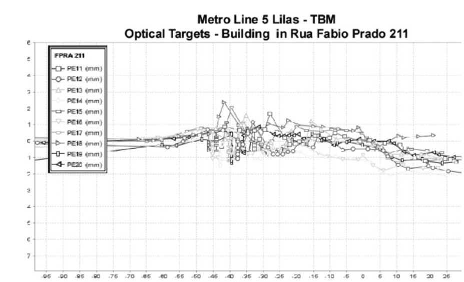

Figure 13. Monitoring lower part of the structural pillars of the building with optical target. PE10 – PE11 (by SACI-METRO system).

in relation to the foreseen limits. The maximum settlement measured was from the tassometer T-01 to the monitoring section EX 21+144 (3,5 mm). The optical target installed at the lower part of the structural pillars of the building has undergone a vertical deformation of 2–3 mm maximum, with angular rotations practically nil (Figures 8, 9, 10, 11, 12).

3 CONCLUSIONS

The standard of the approach used for the PAT improving for the passage of the TBM below the building Commercial-Offices Francisco de Vitoria Street is shown from: i) the continuous observance of the limits of recorded TBM parameters; ii) the monitoring data results, with small vertical deformation of the foundations building (2–3 mm); iii) the absence of building damages.

The PAT improving was the result of a correct analysis approach of the boundary conditions to the tunnel, in particular taking into consideration the contribution of the up-load of the building at the TBM face pressure and its distribution along the section, defined with a pondering on the deformation behavior related to the backfill grouting, variation to the heading, lateral geological structuring and presence of others interferences.

In order to define the up-pressure contribution in the PAT due to footing pressure of the building, the Steinbrenner Chart has been used to get an easy and rapid evaluation of the vertical up-pressure contribution for the tunnel construction step. The choice of the Steinbrenner Chart, based on the theory of elasticity, has been effective considering the geotechnical condition shown, but it is highlighted that in some cases, a model based on the elasticity theory is not a properly valid procedure, as to the case of a very stiff layer above to a less stiff. In such cases the use of a more precise model is recommended.

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