Characterization of Granite and the Underground Construction in Metro do Porto, Portugal.

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Proc. International Conference on Site Characterization, Porto, Portugal, 19-22 September, 2004

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Keywords: Rockmass classification, granite, tunnelling

ABSTRACT: The characterization of the granitic mass of Porto for the design and construction of the Metro works of the city was based on weathering grades and structural features which were used for the derivation of the design parameters. The highly variable nature of the deeply weathered Oporto granite posed significant challenges in the driving of the 2.3 km long C line and the 4 km long S line of the project. Two 8.7 m diameter Herrenknecht EPB TBMs were used to excavate these tunnels but the nature of the rock mass made it extremely difficult to differentiate between the qualities of the mass and apply an open or a closed mode operation of the TBM accordingly. Thus early problems were encountered due to over excavation and face collapse. The matter was finally resolved by the introduction of an Active Support System, which involves the injection of pressurized bentonite slurry to compensate for deficiencies in the face support pressure when driving in mixed face conditions. Both the C and S lines have now been completed with minimal surface subsidence and no face instability.

1. INTRODUCTION

In late 1998 the Municipality of Porto took a decision to upgrade its existing railway network to an integrated metropolitan transport system with 70 km of track and 66 stations. Seven kilometres of this track and 10 stations are located under the picturesque and densely populated city of Porto, an UNESCO world heritage site. A map of the surface and underground routes is presented in Figure 1. Metro do Porto SA, a public company, is implementing the project. The design, construction and operation of this concession were awarded to Normetro, a joint venture. The civil works design and construction was awarded to Transmetro, a joint venture of Soares da Costa, Somague and Impregilio.

The underground tunnel, driven by two Earth Pressure Balance (EPB) TBMs, has an internal diameter of 7.8 m and accommodates two tracks with trains. Line C stretches 2,350 m from Campanhã to Trindade and has five underground stations, a maximum cover of 32 m and a minimum of 3m before reaching Trinidad station. Line S is 3,950m long and runs from Salgueiros to São Bento with 7 stations and a maximum overburden of 21 m.

Figure 1: Map of Metro do Porto routes. Underground tunnels are Line C from Campanhã to Trindande and Line S from Salgeuiros to São Bento.

Tunnel driving was started in August 2000 with the drive from Campanhã to Trindade. It was originally planned that the EPB TBM would be run with a partially full, unpressurized working chamber in the better quality granite in order to take advantage of the higher rates of advance in this mode as compared with operating with a fully pressurized working chamber. It was soon found that the highly variable nature of the rock mass made it extremely difficult to differentiate between the better quality rock masses in which the working chamber could be operated safely with no pressure and the weathered material in which a positive support pressure was required on the face. There were indications of over-excavation and two collapses reached the surface. The second occurred on 12 January 2001, almost a month after the passage of the TBM on 16 to 18 December 2000. This collapse resulted in the death of a citizen in a house overlying the tunnel.

At the invitation of Professor Manuel de Oliveira Marques, Chief Executive Officer of Metro do Porto S.A., one of the authors (E.H) visited Porto from in early February 2001 to review the geotechnical and tunnelling issues of the C Line tunnel. As a result of this visit a Panel of Experts, consisting of the authors of this paper, was established in order to provide advice to Metro do Porto.

2. GEOLOGICAL CONDITIONS

The underground portion of the line passes through the granite batholith which was intruded into the Porto-Tomar regional fault in the late Hercinian period (Figure 2). The Porto Granite, a medium grained two mica granite, is characterized by deep weathering and the tunnel passes unevenly through six grades of weathering and alteration ranging from fresh granite to residual soil. The granite is crossed randomly by aplitic/pegmatitic dykes which display much less weathering, following tectonically determined tension joints.

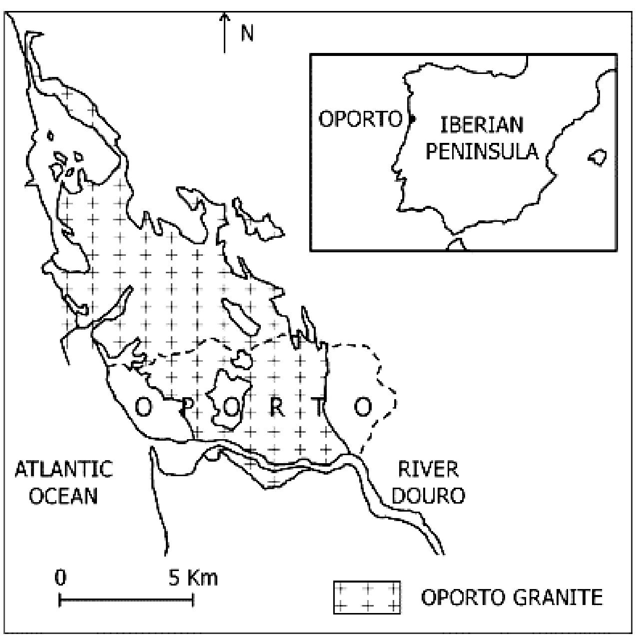
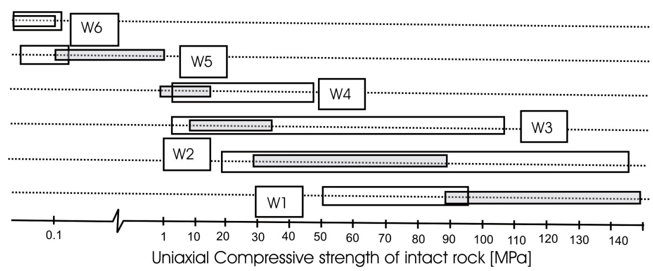


Figure 2: Distribution of granite in the City of Oporto (from A. Begonha and M. A. Sequeira Braga, 2002)

3. CHARACTERISATION OF WEATHERING

The particular feature of most engineering significance of the rock mass is its weathering. All weathering grades (W1 to W6, as established in the engineering geological classification according to the scheme proposed by the Geological Society of London, 1995, and the recommendations of ISRM) can be encountered. Through analyzing the associated geomechanical properties from laboratory tests, the designers developed a re-classification of the degree of weathering aiming to better define the characteristic values of each class and to reduce the overlap between classes (Table 1, Russo et al., 2001).

Table 1: Weathering classes over the uniaxial compressive strength range (clear bars indicate classification based only on qualitative evaluation, shaded bars indicate re-classification after statistical analysis, from Russo et al., 2001)



The depth of weathering is of the order of few tens of meters as weathering was assisted by the stress relief regime due to the deepening of Duro valley. Depths of weathering of 30m are reported by Begonha and Sequeira Braga, 2002. Hence, the ground behaviour varies from a strong rock mass to a low cohesion or even cohesionless granular soil. The granularity and frictional behaviour is retained, as the kaolinitisation of feldspaths is not complete and the clay part not important. Furthermore, the spatial development of the weathered rock is completely irregular and erratic. The change from one weathered zone to another is neither progressive nor transitional. It is thus possible to move abruptly from a good granitic mass to a very weathered soil like mass. The thickness of the weathered parts varies very quickly from several meters to zero. Blocks of sound rock, “bolas”, of various dimensions can “float” inside a completely decomposed granite. Weathered material, either transported or in situ, also occurs in discontinuities.

A particularly striking feature is that, due to the erratic weathering of the granite, weathered zones of considerable size well beyond the size of typical “bolas” can be found under zones of sound granite (see Figure 3). While this phenomenon is an exception rather than the rule and it was expected to disappear with depth, it could not be ignored in the zone intersected by the construction of the metro works. A typical case of such setting is in Heroismo station where weathered granite with floating cores of granite occurs under a surficial part of a sound granitic rock mass (Figure 5).

Figure 3: Appearance of different degrees of weathering in granite in a core recovered from a site investigation borehole on the tunnel alignment. Note that the weathered granite in the left box is at a depth of about 24m under the sound granite of the right box. This must therefore correspond to a huge boulder (core).

Figure 4: Appearance of Oporto granite in the face of an excavation for the new (2002) football stadium. Fracturing of the rock mass and heterogeneity in weathering is obvious

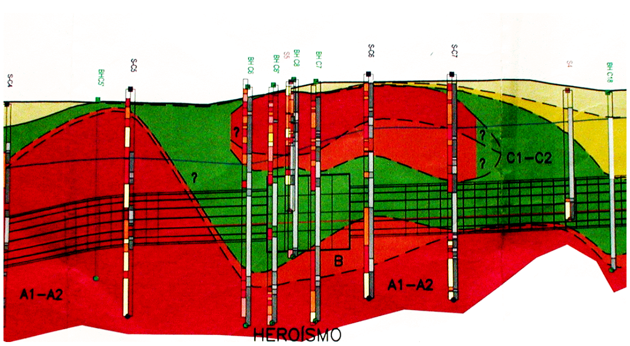


Figure 5: Predicted geology for the Heroismo mined station (Assessment by Transmetro, documents of Metro do Porto). Heterogeneity in weathering and its erratic geometry is evident.

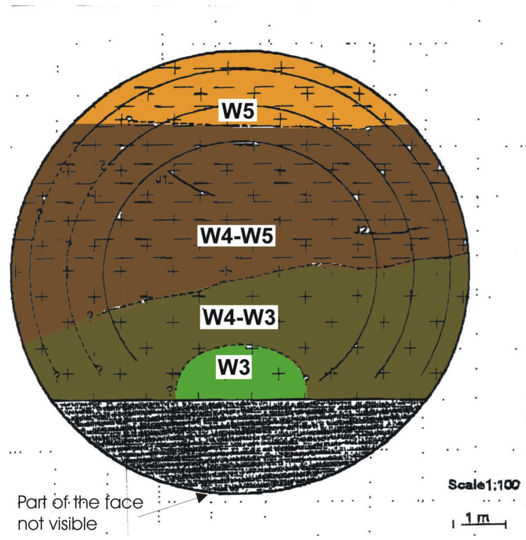


Figure 6: As typical distribution of weathered granite in the face of the EPB driven Tunnel.

4. CHARACTERIZATION OF GRANITIC ROCK MASSES

The definition of rock mass properties for use in the face stability analyses and the machine selection, in the design of stations and the settlement- risk analysis, was based on a geotechnical characterization of the granitic mass in various groups. The approach applied in the design is illustrated in Table 2 and the values of the geotechnical parameters selected after statistical analysis are shown in Table 3 (Russo et al., 2001, Quelhas et al., 2004). Groups g5, g6 and g7 refer to material with soil-like behaviour. Thus it was generally possible to apply principles of soil mechanics to define the geotechnical parameters and the design values of the soil mass were based on sample properties, taking into account the results of the available in situ tests (SPT, etc).

Deformations modulus for groups g2 and g3 was derived from empirical correlations and the results of the 136 Menard tests conducted in the boreholes. It is worth noting that the values of the pressiometric modulus showed significant variability when only associated with the weathering class. On the other hand, when the structure of the mass was considered, variability and discrepancies were significantly reduced (Russo et al., 2001).

It is clear that this characterization cannot be integrated in the design for the selection of parameters, without taking into account the spatial development and variation of geotechnical groups along the alignment or in the area around the stations.

The significance of this comment was shown dramatically soon after boring with the EPB TBM has started. Thus, for the needs of this specific mechanized excavation such a characterization was meaningless and the mode of operation of the TBM had to be selected in such a way that the worst anticipated conditions could be dealt with at any time.

Table 2. Conceptual procedure for the geotechnical characterization of the granitic rock mass and for design (from Russo et al., 2001)

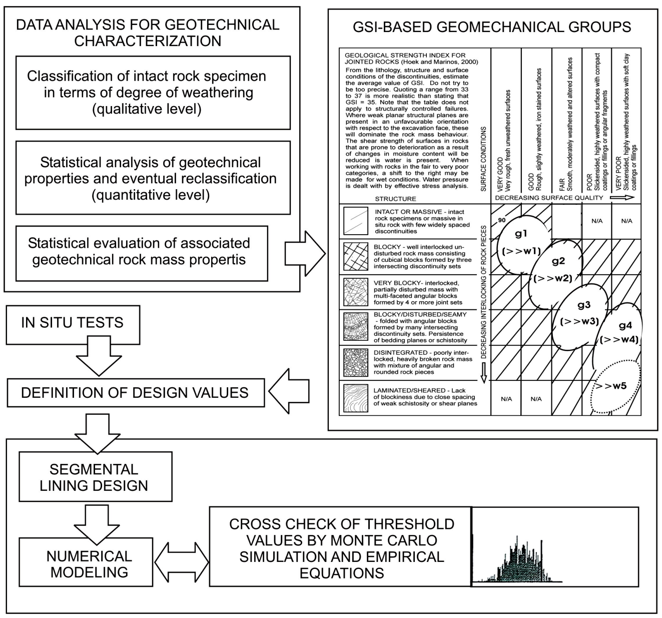


Table 3 Geotechnical parameters (average values, with brackets are given the standard deviations, from Russo et al., 2001 and from Quelhas et al., 2004)

Geotech-nical groups	σ_{ci}	γ (KN/m^3)	Hoek-Brown criterion parameters		Ed (GPa)
			mb	s	
g1	90-150	25-27	7.45 (1.15)	6.9E-2 (3.2E-2)	35 (10)
g2	30-90	25-27	3.2 (0.5)	7.5E-3 (3.4E-3)	10.7 (3.0)
g3	10-35	23-25	0.98 (0.07)	7.5E-4 (1.7E-4)	1.0 (0.5)
g4	1-15	22-24	0.67 (0.12)	0	0.4 (0.2)

Geotechnical groups	N_{SPT}	γ (KN/m^3)	c´ (MPa)	φ´ (°)	Ed (GPa)
g5	>50	19-21	0.01-0.05	32-36	0.05-0.20
g6	<50	18-20	0-0.02	30-34	0.02-0.07
g7	Var.	18-20	0	27-29	<0.05

5. PERMEABILITY

The permeability of the rock mass is dependent upon the weathering grade and the associated fractures. In the less weathered rock the flow is related primarily to the fracture system while, in the more heavily weathered material, the ground behaves more like a porous medium. Porosity in the latter case may have been increased from leaching and this together with the highly variable permeability of the rock mass, has resulted in a very complex groundwater regime. The overall permeability is rather low; of the order of 10^{-6}m/s or lower. However higher permeabilities were measured in pumping tests. We consider that preferential drainage paths exist within the granite mass. The very weathered material, having little or no cohesion, may be erodible under high hydraulic gradients.

The frequent occurrence of old wells connected by drainage galleries was a hazard for tunnelling. Opinion was expressed that long term exploitation of these wells had led to the washing out of fines increasing permeability and formation of an unstable soil structure (Grasso et al., 2003)

6. EPB TBM CHARACTERISTICS

The complex geological and hydrogeological conditions described above resulted in a decision by Transmetro to utilize an 8.7 m diameter Herrenknecht EPB TBM (see Fruguglietti et al. 1999, and 2001). Initially, only one machine was to be used to drive both lines but following start-up problems, a second machine was added in order to make it possible to complete the tunnel drives on schedule.

The TBMs are equipped with a soil conditioning system capable of injecting foam, polymer or bentonite slurry into the working chamber. Muck removal is by continuous belt conveyor from the TBM back-up to the portal and then by truck to the muck disposal areas. Tunnel lining is formed from 30 cm thick, 1.4 m wide pre-cast concrete segments. The lining comprises six segments and a key and dowel connectors are used in the radial joints while guidance rods are used in the longitudinal joints. The features of the EPB TBM are illustrated in Figure 7. In a review paper by N. Della Valle (Tunnels and Tunnelling, 2002) details are presented. Gugliementi et al. (2004), in a recent paper, offer a full presentation of the control of ground response and face stability during excavation. In those papers issues proposed by the authors of the present paper and discussed here are described.

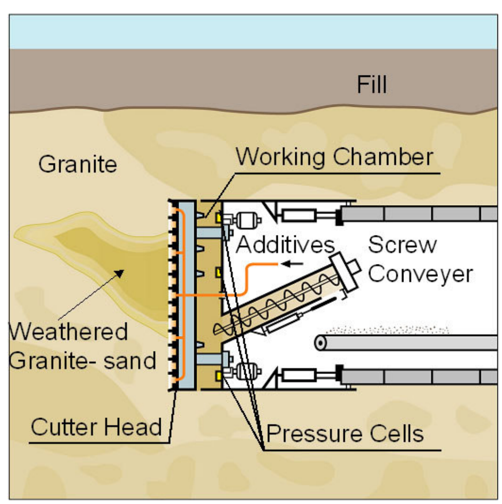


Figure 7: Characteristics of the Herrenknecht EPB TBM used in Oporto.

7. CHARACTERIZATION OF GEOLOGICAL CONDITIONS IN TERMS OF THE TBM OPERATION

The geological conditions discussed above can be translated to the following geological models in front, at the face and immediately above the TBM:

1. Granitic mass of sound or slightly weathered rock, no weathered material in the discontinuities;
2. Granitic mass of sound or slightly weathered rock but with very weathered material (filled or in situ) in substantial fractures; these fractures may communicate with overlaying parts of completely weathered granite;
3. Very weathered or completely weathered granite, W5 (almost granular soil with little or no cohesion);
4. Very weathered or completely weathered granite with blocks of the rock core;
5. Mixed conditions with both sound mass and completely weathered granite appearing in the face.

In all cases the water table is above the tunnel crown

Only the first of these geological models can be excavated using an EPB TBM operating in an open mode. However, because of the unpredictable changes in the geological conditions described above, we considered that the risk of operating in an open mode was unacceptable unless there was unambiguous evidence that this condition persisted for a considerable length of tunnel drive. This was not the case in this tunnel and we recommended that the entire drive should be carried out with the TBM operating in a closed mode.

Indeed in all other models, uncontrolled over-excavation could occur unless the chamber of the machine was full of appropriately conditioned excavated material with the necessary support pressure and control of the evacuation of the muck through the screw conveyor. Lack of adequate face support could result in piping of the weathered material in the fractures that could, in turn, induce collapse of the overlying weathered granite. The mixed face conditions described in item 5 above were considered to be particularly difficult because of the uneven pressure distribution on the face induced by the different stiffness of the rock and soil masses. The successfully handling of this problem is discussed in a following section.

A significant number of wells and old galleries exist in the area and, while most were located on old city maps and by inspection of existing properties, there remained the possibility that some unpredicted wells and galleries could be encountered. The wells usually end above the tunnel but some were deep enough to interfere with the construction. The crossing of such features clearly involved some risk but this was substantially lower when operating the TBM in a fully closed and pressurised mode than in an open or partially open mode.

6. FACE SUPPORT PRESSURE

The face support pressure of EPB - TBMs was controlled by measuring the pressure at the bulkhead with pressure cells, approximately 1.5 m from the face, as shown in Figure 8. In closed mode operation, the working chamber is completely filled with conditioned excavated material, the earth paste. The earth paste is pressurized by the advancing forces induced by the advance jacks via the bulkhead. The pressure level is controlled by the effectiveness of the excavating cutter head in relation to the discharging screw conveyor.

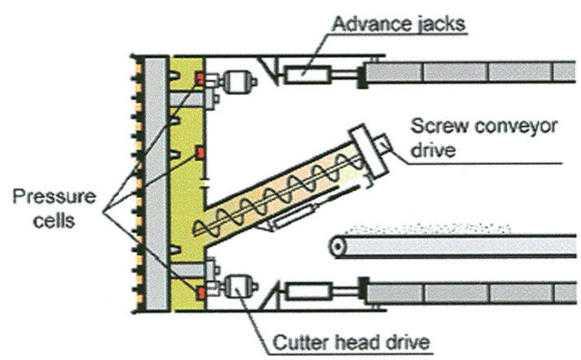


Figure 8: Measurement devices for face support pressure

To verify complete filling of the working chamber, the density of the earth paste in the working chamber was controlled by pressure cells on the bulkhead at different levels. This method satisfies the demand of preventing a sudden instability of the face caused by a partially empty working chamber but it does not guarantee a reliable face support pressure.

Pressure measurement at the bulkhead, 1.5 m behind the face, provides only partial information about the support pressure at the face. The support medium, the earth paste created from excavated ground, conditioned by a suspension with different additives, must have the physical properties of a viscous liquid. However, the shear resistance in that viscous liquid reduces the support forces which can be transferred onto the face. The shear resistance of the earth paste depends on the excavated ground and the conditioning, which is a complex and sensitive procedure. Consequently, the shear resistance of the support medium often varied considerably.

Therefore, the fluctuation of the face support pressure could exceed 0.5 bars. This fluctuation may be acceptable in homogeneous geology but in mixed ground, as found in the Oporto granite, the variable support pressure entailed the danger of significant over excavation.

One of the processes which can cause a drop in the face support pressure is illustrated in Figure 9 which shows a situation in which the lower part of the face is in unweathered granite while the upper part of the face is in residual soil. A major part of the thrust of the machine is consumed by the cutter forces required to excavate the unweathered granite and there is a deficiency in the forces available to generate the pressure in the earth paste in the upper part of the working chamber. This results is an imbalance between the soil and water pressure in the unweathered granite and the support pressure in the upper part of the working chamber. If this deficiency is too large, the face will collapse inwards into the working chamber and this will result in progressive over excavation ahead and above the face.

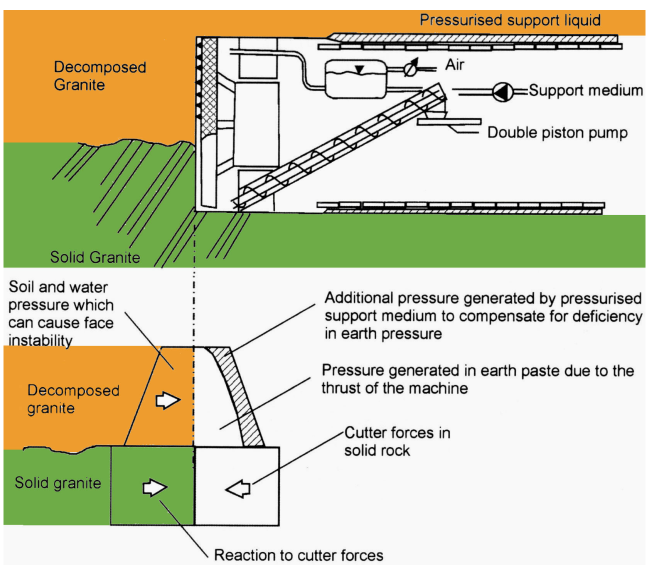


Figure 9: Face support pressures in mixed face conditions in Oporto granite. An Active Support System for overcoming the support pressure deficiency is also illustrated.

The deficiency of face support pressure can be compensated for by the addition of an Active Support System, proposed by Dr Siegmund Babendererde (one of the authors of this paper) and shown in Figure 9. This system is positioned on the back-up train and consists of a container filled with pressurized bentonite slurry linked to a regulated compressed air reservoir. The Bentonite slurry container is connected with the crown area of the working chamber of the EPB TBM. If the support pressure in the working chamber drops below a predetermined level, the Active Support System automatically injects pressurized slurry until the pressure level loss in the working chamber is compensated. The addition of this Active Support System to the EPB TMB results in an operation similar to that of a Slurry TBM. This automatic pressure control system reduces the range of fluctuations of the face support pressure to about 0.2 bar.

In the case of an open and potentially collapsible structure in the weathered granite surrounding the wells, resulting from leaching of the fines, we considered that stable face conditions can be maintained by the correct operation of the TBM in fully closed EPB mode with supplementary fluid pressure application. However, care was required in the formulation and preparation of the pressurizing fluid in order to ensure that an impermeable filter cake was formed at the face. This was necessary in order to prevent fluid loss into the open structure of the leached granite mass.

The application of the Active Support System in the Metro do Porto project was the first time that this system had been used. There was initial concern that the addition of the bentonite slurry would alter the characteristics of the muck to the point where it could no longer be contained on the conveyor system and that an additional slurry muck handling facility may be required. This concern proved to be unfounded since the volume of bentonite slurry injected proved to be very small and there was no discernable change on the characteristics of the muck.

The predetermined support pressure was determined from calculations using the method published by Anagnostou and Kovari (1996) which proved to be reliable for these conditions. The Active Support System was extremely effective in maintaining the predetermined support pressure and no serious face instability or over excavation problems were encountered after it was introduced. In fact, the system permitted the 8.7 m diameter tunnel to pass under old houses with a cover of 3 m to the foundations, without any pre-treatment of the ground. Surface settlements of less than 5 mm were measured in this case. The boring of the section under this shallow cover is described in a paper of Diez and Williams, 2003.

The Active Support System was also connected to the steering gap abound the shield and the filling of this gap with bentonite slurry provided a reliable means of maintaining a predetermined pressure in this gap.

CONCLUSIONS

The highly variable characteristics of the weathered granite in Oporto and their sudden changes imposed substantial risks on the driving of the C and S lines by means of EPB TBMs. The impossibility of accurately predicting and maintaining the correct face support pressure resulted in significant over excavation and two collapses to surface during the first 400 m of the C line drive. Characterization in different geotechnical groups for the selection of the mode of operation of the EPB was almost meaningless and the mode of operation of the TBM had to be selected in such a way that the worst anticipated conditions could be dealt with at any time.

The introduction of the Active Support System, which involves the injection of pressurized bentonite slurry to compensate for deficiencies in the face support pressure when driving in mixed face conditions, proved to be a very effective solution. The remaining C and S line drives have now been completed without further difficulty although the rate of progress was less than that originally projected when the project was planned.

The final breakthrough of the C line drive is illustrated in Figure 13.

Figure 13: Final breakthrough of the TBM S-203 on the completion of the drive from Salgueiros to Trindade on Thursday 16 October 2003.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the permission of Metro do Porto to publish the details contained in this paper. The cooperation of Transmetro and particularly of Ing. Giovanni Giacomin in working with the Panel of Experts is also acknowledged. Part of this paper was presented in a workshop in Aveiro, Portugal, April 2004.

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