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A TUNNEL COLLAPSE ON THE CONSTRUCTION IN METRO DO PORTO: SOLUTIONS FOR OPTIMIZATION OF ADVANCE CONTROL PARAMETERS OF A EPB TBM

AWARIA TUNELU PRZY BUDOWIE METRA W PORTO: ROZWIĄZANIA DLA OPTYMALIZACJI ZAAWANSOWANYCH PARAMETRÓW STEROWANIA MASZYNY DRĄŻĄCEJ TUNELE EPB TBM

Abstract The development of an important project of the new Metro of Porto urban area has mobilized a significant effort in order to have a good characterization of the involved ground features. The highly variable nature of the deeply weathered Porto granite posed significant challenges in the driving of the two tunnels of 6.5 km constituting the main lines under the historical centre. Two 8.7 m diameter 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. These occurred in some problems due to over excavation and face collapse, ending in a serious collapse that imposed a redesign of the equipment and process, by introducing an Active Support System, involving the injection of pressurized bentonite slurry to compensate for deficiencies in the face support pressure when driving in mixed face conditions. The variability of these weathered masses has great influence in the geotechnical solutions and construction procedures, demanding for special solutions and unusual control systems and safety protocols, which enabled a excellent development of the construction after the accident, with high productivity and zero accidents.

Streszczenie Realizacja ważnego projektu Nowego Metra w aglomeracji Porto spowodowała mobilizację znacznych wysiłków, mających na celu uzyskanie dobrej charakterystyki podłoża gruntowego. Bardzo zmienny charakter głęboko zwietrzałego granitu Porto był ogromnym wyzwaniem podczas wiercenia dwóch tuneli o długości 6.5 km, stanowiących główne kierunki metra pod historycznym centrum miasta. Dwie Maszyny Drążące Tunele EPB TBM o średnicy 8,7 m, zostały użyte do wydrążenia tych tuneli, ale charakter górotworu niezwykle utrudnił odróżnienie jakości górotworu i zastosowanie odpowiedniej techniki głębienia tuneli systemem otwartym lub zamkniętym. Fakt ten spowodował określone problemy, które wywołały zawalenie się wyrobiska i utratę stateczności czoła tunelu, zakończonego poważną awarią. Skutkiem awarii było przeprojektowanie urządzeń i procesów technologicznych, poprzez wprowadzenie aktywnego systemu wspierającego (ASS), obejmującego wtrysk zawiesiny bentonitu pod ciśnieniem, aby zrekompensować braki w części czołowej tunelu, które wystąpiły podczas wiercenia w czołowej części tunelu. Zmienność zwietrzałych mas skały miała ogromny wpływ na rozwiązania geotechniczne i procedury budowlane, wymagające specjalnych rozwiązań i nietypowych systemów kontroli oraz przygotowanie protokołów bezpieczeństwa. Wprowadzenie tych procedur umożliwiło dalszy doskonały rozwój budowy po wypadku, z dużą wydajnością przy zerowym poziomie wypadków.

1. Introduction

From 1998 to 2004 the city of Porto has assisted to a „revolutionary" renew of the transportation infrastructures, by upgrading its existing railway network to an integrated metropolitan transport system with 70 km of track and 66 stations. Seven kilometers of this track and 10 stations were constructed in the very first phase under the historical and densely populated city, an UNESCO world heritage site. A map of the surface and underground routes is presented in Fig. 1 (Babendererde et al., 2004). The public company that rules all the planning, construction process and the exploration of the new lines is Metro do Porto SA (www.metrodoporto.pt/en/). The design, construction and operation concession of the first phase 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 Impregilo.

The underground tunnels were driven by two Earth Pressure Balance (EPB) TBMs, with an internal diameter of 7.8 m and 8.0 m to accommodate two tracks with trains. Line C develops in tunnel for 2,350 m from Campanhã to Trindade and has three underground stations, a maximum cover of 32 m and a minimum of 3 m before reaching Trindade station. Line S is 3,950 m long and runs from Salgueiros to São Bento with 7underground stations and a maximum overburden of 21 m.

Fig. 1. Map of Metro do Porto routes. Underground tunnels are Line C from Campanhã to Trindade and Line S from Salgueiros to São Bento (Babendererde et al., 2004)

As described by Babendererde et al. (2004), tunnel construction started in August 2000, driving from Campanhã to Trindade, and was originally planned that the EPB TBM would 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. The highly variable nature of the rock mass, as described in what follows, 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 (Babendererde et al., 2004).

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 (Fig. 2).

Fig. 2. Accident on 12 January 2001; the collapse resulted in the death of an old lady in a house overlying the tunnel; a sink hole was induced after 30 m of the passing of the face

After this accident the administration of Metro do Porto invited a Panel of Experts, Dr. Siegmund Babendererde, from Germany, Dr. Evert Hoek, from Canada, Prof. Paul Marinos, from the National Technical University of Athens, in Greece, and Prof. António Silva Cardoso, from the University of Porto, in order to provide advice to Metro do Porto.

2. Interface Geomaterials or Hard Soils – Soft Rocks and Residual Soils

Interface geomaterials (IGM) or „Hard Soil - Soft Rock" (HSSR) are materials that lie on the boundary between soils and rocks. Terzaghi and Peck (1967) defined hard soils as those with an unconfined compressive strength in excess of 400 kPa. As defined by Clayton et al. (1998), „soil is an aggregate of mineral grains that can be separated by such gentle mechanical means as agitation in water", while rock was described as „a natural aggregate of minerals connected by strong and permanent cohesive forces". Current practices define the strength of the strongest soils and the weakest rocks using a variety of terms and simple hand and laboratory tests, but there is usually a gap in these definitions between the weakest rocks and the strongest soils (Clayton et al., 1998).

While in British Standard (BS5930:1981) the strongest soils are termed „very stiff or hard" and are recognised by an undrained strength > 150 kPa, approximately equivalent to an uniaxial unconfined compressive (or simply compression) strength, qu (or UCS) of about 0.3 MPa, the weakest rocks are termed „very weak", and have a uniaxial compressive strength of less than 1.25 MPa. In the USA (ASTM D2488-90) this term „hard" is equivalent to the British term „very stiff".

There is a very „fluid" difference or similarity in the concept of Intermediate Geomaterials (IGM) and residual soils, a natural product of extreme weathering of rock masses. As stated by Cruz (2010), beyond the first three weathering degrees of ISRM classification (W1 to W3), chemical weathering is extended to the whole massif, and so the mechanical evolution is mainly governed by increasing material porosity, the weakening of mineral grains and the reducing bonding between grains, with the rock massif becoming more and more friable and chemically weakened. Weathering degrees W4 and W5 represent transition behaviour where micro and macro fabrics have similar influence, towards a residual soil-mass where the relict macro-fabric is no longer present. This process is followed by a mechanical degradation that leads to substantial reduction of strength and stiffness. Table 1 illustrates orders of magnitude typically associated to rock and soil-masses of some reference parameters.

Table 1 – Geotechnical parameters (Babendererde et al., 2004)

	UCS (q_u) MPa	***c′* (MPa)**	***Ed* (GPa)**
Rock	2 - 300	> 0,1	>400
Soil	< 2	< 0,1	< 300

UCS (q_u) – uniaxial compression strength; c' – effective cohesion intercept;
Ed – Young Modulus

When macrofabric is no longer represented, then a general cohesive-frictional behaviour of soil takes place, with the overall mechanical behaviour governed by a wide range of factors such as micro-structure, stiffness non-linearity, small and large strain anisotropy, weathering and de-structuration, consolidation characteristics and flow rate dependencies (Schnaid, 2005). The author emphasized that IGM or residual soils satisfy at least one of the following criteria:

– Classical constitutive models do not offer a close approximation of its true nature;
– It is difficult to sample or to be reproduced in laboratory;
– Very little systematic experience has been gathered or reported;
– Values of geomechanical parameters are outside the range expected for more common sands and clays;
– The soil state is variable due to complex geological conditions.

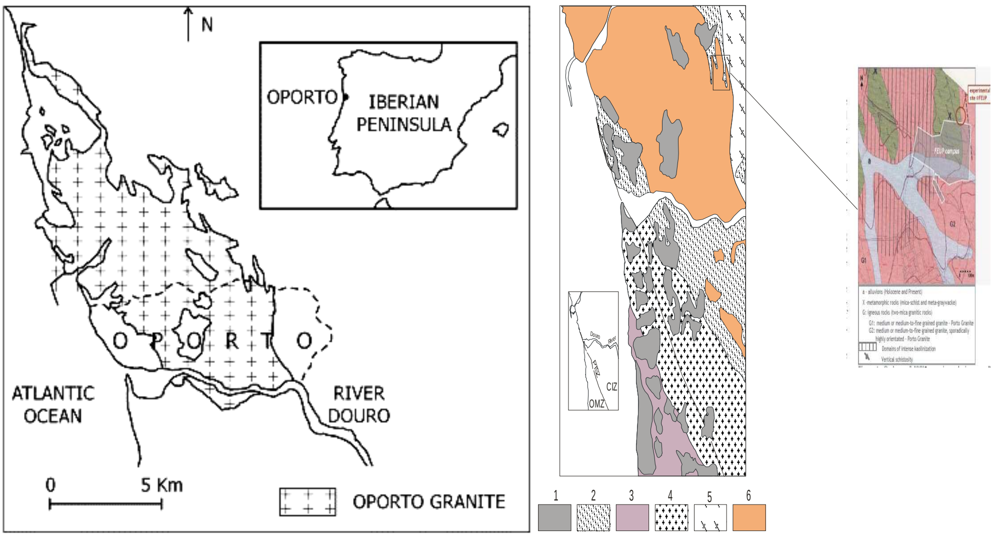
3. The complex Geology of Porto

3.1 Introduction

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 (Fig. 3). 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.

The north region of Portugal is largely dominated by residual soils from different nature, namely the weathering profiles of granite rocks are well known by the presence of very sound rock blocks („boulders”) involved into highly-weathered material. Weathering profiles of Porto granitic masses are very complex. They are mostly, but not always, characterized by its gradation from upper weathered levels to lower sound rock, improving its mechanical behaviour with depth. Meanwhile, the weathered granite zones present a wide irregularity, with common blocks of sound or partly weathered granite, with various dimensions involved in high to extremely high weathered masses.

The presence of those granite blocks is related to geological structures such as fractures, foliation or faults, where water can easily flow into and through, accelerating chemical weathering processes. They can be reasonable determined when a careful and rigorous geological-geotechnical site investigation campaign involving the all excavated volume (both in area and depth), is carried out into the boreholes net, in order to construct a reliable geological-geotechnical model. During the Porto Metro construction this methodology has conducted to very good results, but when applied to the prediction of rock blocks or the position of boundaries between different rock weathering classes, in areas located outside the borehole net, several problems occur.



CIZ – Central Iberian Zone; MZ – Ossa-Morena Zone; PTSZ – Porto-Tomar Shear Zone; (1) Cenozoic cover deposits; (2) Cambrian – CIZ (Schist and Graywacke Complex); (3) Precambrian metassediments – OMZ; (4) Madalena granite (late orogenic); (5) Ermesinde granite (hybrid sin-tectonic); (6) Porto granite (peraluminous sin-tectonic)

Fig. 3. Distribution of granite in the City of Oporto (Begonha & Sequeira Braga, 2002)

3.2 Characterization of Weathering

Several researchers have been studying igneous rock weathering profiles all over the world. The works by Little (1969), Vargas (1971), Deere & Patton (1971), Martin & Hencher (1986) and also the technical committees from IAEG (1981) and ISRM (1981) assumed very important roles in the process. In Portugal Begonha (1997) and Viana da Fonseca (1996) have developed some comprehensive works on the weathering profiles of regional igneous rocks.

In Porto region, 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 Douro valley. Depths of weathering of 30 m are reported by Viana da Fonseca, 1996, Begonha, 1997, 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 is not important. Furthermore, the spatial development of the weathered rock is completely irregular and erratic (Babendererde et al., 2004).

These weathering profiles are usually characterized by a decrease in weathering degrees from upper levels to deeper sound rock, with inherent improvement of its geomechanical properties with depth. Weathered granite zones are commonly very irregular in extension and intensity of weathering. It is also common to occur sound or slight weathered rock blocks involved by soil or high weathered masses. According to Begonha (1997) and Marques et al. (2001), the occurrence of erratic weathering grades is common in the Porto Granite masses, varying from sound rock, with no sign of weathering (W1, on ISRM's classification), up to residual soil (W5). Further information on Oporto granite geology can be found in Viana da Fonseca (2002) and Viana da Fonseca et al. (2003). It is 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 completely decomposed granite. Weathered material, either transported or in situ, also occurs in discontinuities.

The change from one weathered zone to another is neither progressive nor transitional. 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 Fig. 4).

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 Heroísmo station where weathered granite with floating cores of granite occurs under a superficial part of a sound granitic rock mass (Fig. 5).

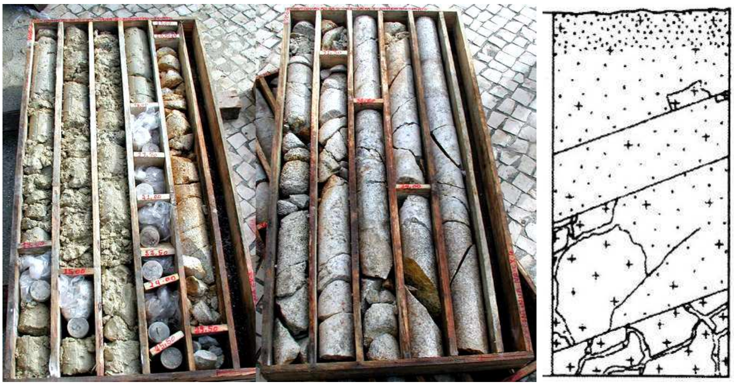
Weathering grades (W1 to W6, as established in the engineering geological classification according to the scheme proposed by the Geological Society of London, 1995, and the recommendations of ISRM) were adopted by several designers following the geomechanical unities identified and used in the geological-geotechnical modelling, in order to represent materials with similar geomechanical characteristics: W5, W5-W4, W4-W5, W4, W4-W3, W4-W3, W3-W4, W3, W3-W2, W2-W3 and W2.

Viana da Fonseca et al. (2003) have presented detailed description of these materials, which is not the scope of this paper. In order to characterize the rock materials that would be encountered in excavated masses, several discontinuities parameters were described, even where only qualitative data was available: orientation, aperture, persistence, permeability, joint roughness and the presence and type of filling material. The paper from Viana da Fonseca et al. (2003) presents the average characteristics for each of these parameters, taken as subclasses of the main ones for ISRM patterns. In this paper, only qualitative data will be indicated.

3.3 Geological and Geotechnical Profiles

In order to construct the geological-geotechnical 3D model of the excavated masses of stations and other subsidiary underground works, a large number of profiles were built. Viana da Fonseca et al., 2003 and Marques et al., 2004 present some of these from Trindade e Aliados stations that are shown in Figure 6.

Others designers of the different geotechnical projects involved in the construction, as described by Babendererde et al. (2004), when analyzing the associated geomechanical properties from laboratory tests, made 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 (Fig. 7, Russo et al., 2001).

a) 

b) 

c) 

Fig. 4. Profiles of highly weathered to sound granitic masses: a) different degrees of weathering in granite in a core recovered from a borehole on the tunnel alignment (weathered granite in the left box is at a depth of about 24 m under the sound granite of the right box – boulder or core); b) Trindade station: inverse weathering in depth, rock cheminees and cut with upper level in rock masses overlaying highly weathered kaolinized horizons. c) new FCP stadium – fracturing of the rock mass and heterogeneity in weathering is obvious

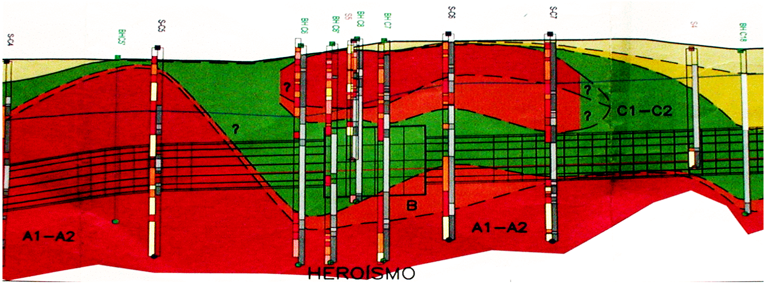
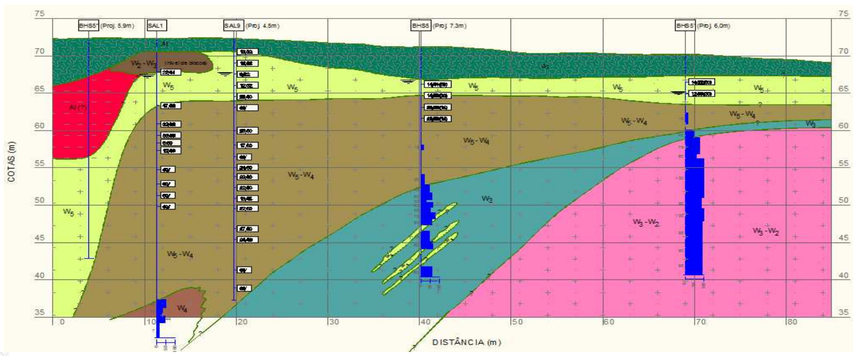


Fig. 5. Predicted geology for the Heroísmo mined station (Assessment by Transmetro, documents of Metro do Porto). Heterogeneity in weathering and its erratic geometry is evident

a)



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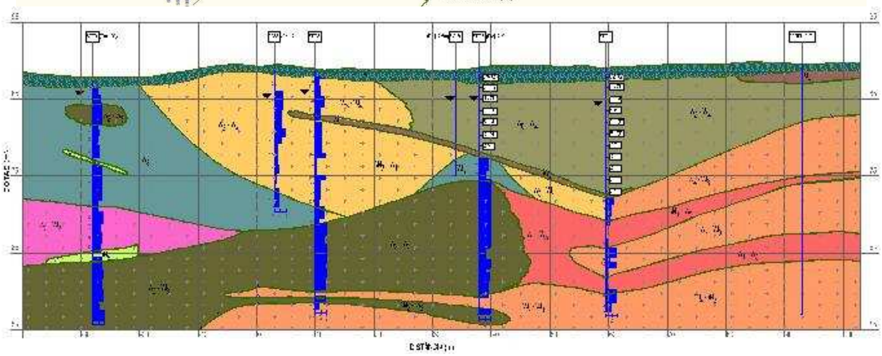

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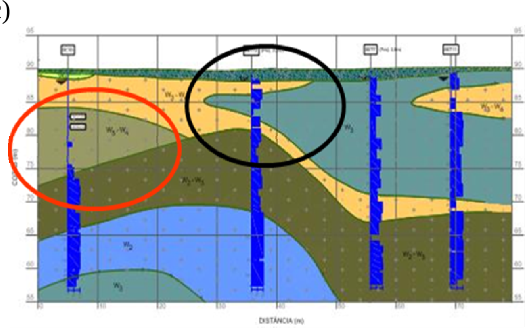

d)

Fig. 6. Geological-geotechnical profiles: a) East and West side of Aliados Station; b) Centre-longitudinal axis in Trindade Station; c) East side of Trindade Station, with the indexation to particular features observed in the cuttings

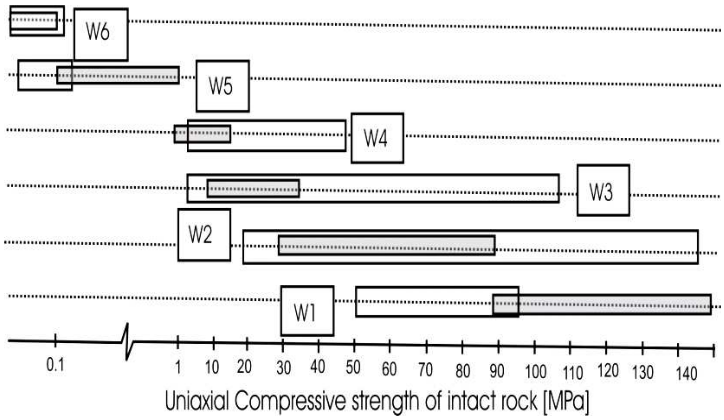

Fig. 7. Weathering classes over the uniaxial compressive strength range (clear bars indicate classification based only on qualitative evaluation, shaded bars indicate re-classification after statistical analysis, from Russo et al., 2001)

3.4 Main Characteristics of Rock Weathering Grades W_4–W_2

Geomechanical characterization was based, as said before, on semi-empirical geomechanical classification proposed by Bieniawsky (1976, and updated in 1989). The summary of uniaxial compressive strength ($UCS = q_u$), from several characterization campaigns, allowed an estimative for each weathering grade, presented on Table 2. For W5 class, $UCS = q_u$, varied between 30 and 140 kPa; but if W5–W4 class are considered, a range between 2 and 5 MPa is obtained. These values were used to estimate geotechnical parameters that were considered in design (Table 2). The unfavourable orientation of discontinuities was considered on these parameters.

Table 2. RMR and UCS = q_u values by rock mass classes

	W2	**W2–W3** **W3–W2**	**W3**	**W3–W4** **W4–W3**	**W4**
$UCS = q_u$		55 MPa	35 MPa	20 MPa	15 MPa
RMR	55	45	25	20	13

3.5 Hydrogeological Conditions

Water level was defined on each borehole of the several campaigns, being always detected close to surface. Water level variation during the year is low (2 m in average) and can be explained by the high water recharge due to the presence of regional aquifers.

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.

A great number of 5 stage Lugeon tests were done in order to evaluate hydrogeological conditions of rocky layers varying between W3–W2 to W4 to W5 classes.

Lefranc tests were carried out on the most weathered horizons (W5–W4 and W5). All results were treated in order to estimate the permeability to be used on percolation model they are presented on Table 3. These values that are obtained for hydraulic conductivity show that the most weathered horizons, W5 class, particularly rich in kaolin present moderate to low permeability values (close to 10^{-7} m/s^2 for short periods of time and close to 10^{-8} m/s^2 for longer ones), while less weathered masses but very fractured (W3, W2 with fracturing F4 to F5) are quite permeable (with values up to 10^{-5} m/s^2), being this controlled by discontinuities.

This characterization of hydraulic conductivity was mainly done by analytical interpretation of the results obtained in pumping tests using radial piezometer control. All these permeability values are relatively low giving rise to a flow of underground water that can be drained by conventional drainage systems. Only on restricted zones the flow may be concentrated and demandable of specific measures to avoid any leakage.

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)

Table 3. Values of hydraulic conductivity for each Oporto Granite weathering class

Weathering classes(W_i – ISRM, 1981)	**Permeability, k (m/s)**	
	References (1)	**Experimental results (2)**
Soil without relict structures (W_6)	Low	$\approx 10^{-7}$
Completely weathered rock – saprolitic (residual) soil (W_5)	Medium	$10^{-6} \div 10^{-5}$ (3)
Very weathered rock (W_4) and fractured (F_4–F_5)	High to medium	$10^{-5} \div 10^{-4}$ (3)
Moderately weathered rock (W_3) and fractured (F_3–F_4)	Medium to high	$10^{-5} \div 10^{-6}$
Slightly weathered rock (W_2)	Medium	$10^{-6} \div 10^{-7}$

(1) Qualitative tendencies: Deere & Patton (1971), Dearman (1976), Costa-Filho & Vargas Jr (1985)
(2) Values obtained from pumping tests with radial piezometer control
(3) Soil matrixes richer in kaolin present permeability one order lower in logarithmic scale

3.6 An Example of an Adjustment to Model Due to Unpredictable Problems

Viana da Fonseca et al. (2003) and Marques et al. (2004) describe a case where the strong inhomogeneity of Porto Granite weathering profiles has proved to be a key feature on geotechnical modelling. This is what was found during the excavation of Aliados Metro Station. Differences in the geotechnical modelling were caused by wrong geological identification and insufficient survey.

The geological model of Aliados station is very complex, as can be seen on Figure 6a and b. Before the last site investigation campaign, which preceded the excavation and the execution of the retention structures, the geological-geotechnical modelling for the masses developing on the extreme west showed the presence of a very thick W3 layer occurring close to surface, which allowed the adoption of a structural solution based on a passive retention system composed by a net of rock bolting eventually complemented by an active solution (Munich type wall with anchors).

The purpose of a new borehole (SAL11) was to investigate a zone where no previous boreholes had been executed and where geologists and civil engineers had some doubt in the interpretation. This caused a redefinition of the geotechnical model, indicating an apparently clear weathered rock mass mainly composed by W4, W5–W4 and W3–W4 units instead of the thick W3 mass previously interpreted. The former (vs 1) and the new model can be visualized in Fig. 8.

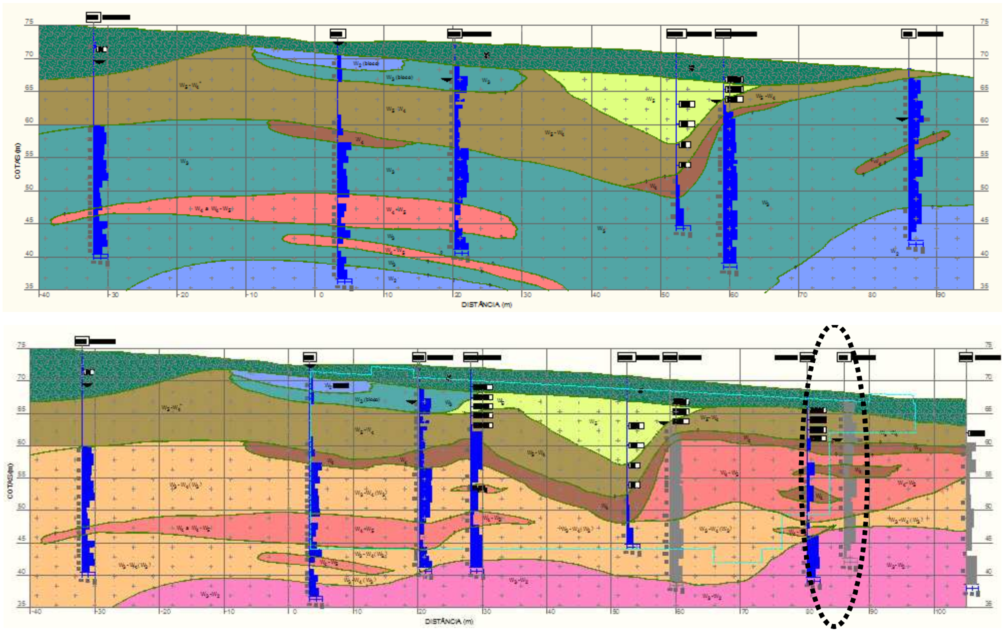

Fig. 8. Geological-geotechnical model for profile AA', extreme western longitudinal boarder of Aliados Station after revision with data from a new borehole (SAL11), marked by an ellipse

This new model, which construction was based on a very detailed description of rock materials, was then used to determine the „final version" of the retaining system designed to stabilize this part of the station („secant piles with multilevel anchorage"). The paradox reality verified during excavation revealed a geological-geotechnical model close to the first one (constructed without SAL11 data), in particular throughout the zone described above, where moderately weathered zones should occur in the revised model. The former model for this zone, mainly based on data from SCS3A/BHS4 boreholes, already predicted this W3 layer. But the final version, based on the integration of new data on the complementary campaign, has caused modifications to this model. Nevertheless this was not at all verified during excavation. In fact, borehole SAL11 was executed along specific discontinuity (probably a faulty zone), and conducted the designers (engineers and geologists) to an erroneous solution.

Seismic tests could have been used to solve this problem but this option was not accepted at that time, by time pressure. The engineering solution adopted to stabilize the excavation walls was to execute the bore piles until the base of a moderately weathered rock layer where a double level of anchorages was installed. Figure 9 shows an illustration of this final transition. The excavation was concluded by this time (drilling for nailing system on the moderately weathered rock mass is seen).

Figure 9. Overall view of the West wall and low weathered rock zone located at South part of this side

The example that has been presented of the geological-geotechnical model, constructed to Porto Metro, has pointed out some important measures to be taken when working in areas dominated by weathering profiles from (eruptive) rocks these are:

- The site investigation campaign must encompass the excavation area inside the borehole net;
- Spacing between boreholes should not be bigger than 20 m;
- A comprehensive knowledge of local hydrogeological model;
- Use of seismic logging between boreholes, or other, in order to confirm geological-geotechnical interpretation from boreholes data must be considered;
- Detailed description of core samples, including identification of transitional materials is needed (W3–W2, W4–W5, etc.).

3.7 Characterization of Granitic Rock Masses for Tunnelling

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 (Babendererde et al., 2004). The approach applied in the design is illustrated in Table 4 and the values of the geotechnical parameters after statistical analysis are shown in Table 5 (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 pressuremeter 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). Babendererde et al. (2004) represent a typical distribution of weathered granite in the face of the EPB driven tunnel (Fig. 10).

Table 4. Conceptual procedure for the geotechnical characterization of the granitic rock mass and for design (from Russo et al., 2001)

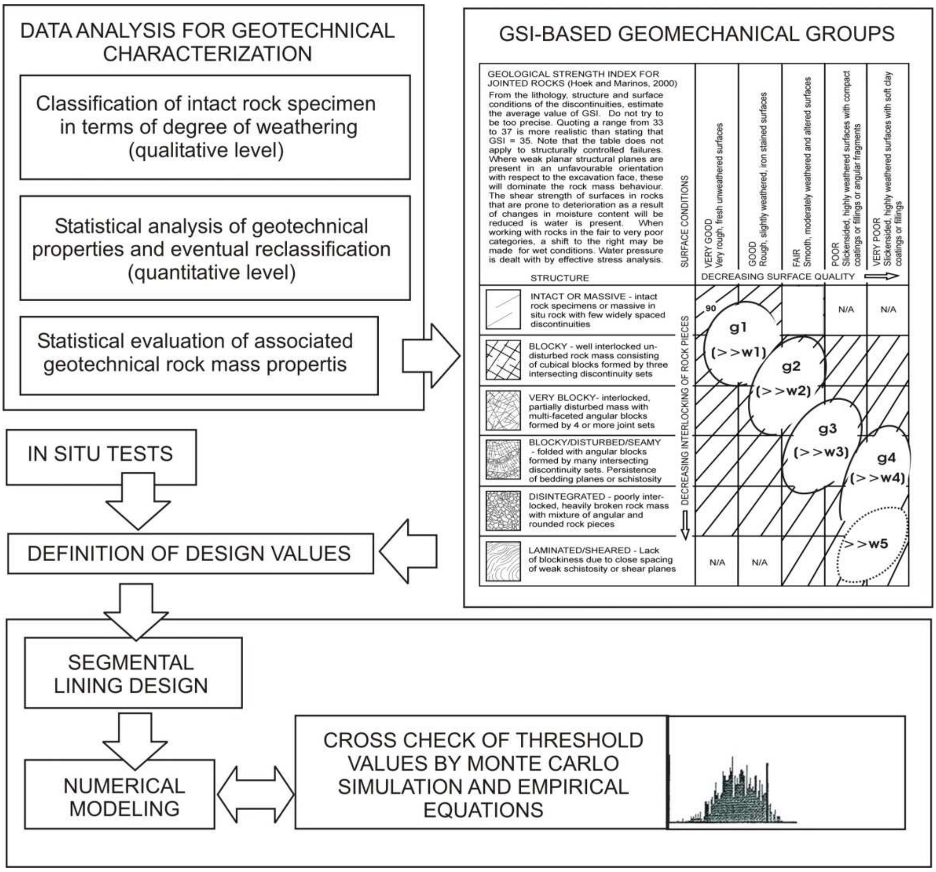

Table 5. Geotechnical parameters (Babendererde et al., 2004)

Geotechnical groups	σ_{ci}	γ (KN/m^3)	Hoek-Brown criterion		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)	**Mohr-Coulomb criterion**		**Ed (GPa)**
			c' (MPa)	ϕ' (^{0})	
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		27÷29	<0.05

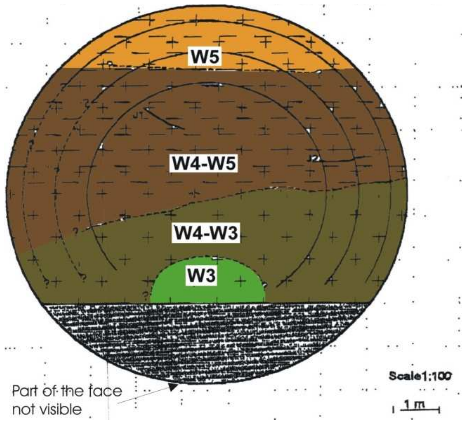

Fig. 10. A typical distribution of weathered granite in the face of the EPB driven Tunnel

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 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.

The large variability of hydraulic conductivity, already referred inducing in the very weathered material, with little or no cohesion, risk of to be erodible under high hydraulic gradients and the frequent occurrence of old wells connected by drainage galleries was a hazard for tunnelling (Babendererde et al. 2004). 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).

4. Tunnelling With a Boring Machine – Geologic Criteria for TBM Selection

The 7.8 m and 8.0 m i.d. tunnels are both lined with a 1.4 m long, 300 mm thick, tapered universal ring, consisting of 6+1 pre-cast, reinforced concrete segments. Rings allow a minimum radius of 200 m.

The ground conditions are resumed by Gaj et al. (2003) having in an overburden thickness ranges from 10 m to 30 m, with a minimum value of 3÷4 m in two sections of the city's historical centre where the TBM has been driven underneath sensitive buildings, with a densely populated urban environment (more than 2000 buildings in the influence zone, including important and historical buildings such as the Town Hall). A specific risk assessment was performed for input into the design of preventive measures. A GIS-based monitoring system controlled the effects induced by tunnelling and to activate the countermeasures (Valdemarin et al., 2002).

„Granito do Porto" formation dominates, with some alluvial basins associated to the presence of several water courses, most of which are covered due to the intense urbanization of the area. The high variability of these formations, their metastability with high collapse potential, due to high porosity and reduced cohesive strength, gives rise to an elastic-brittle-plastic behaviour that may lead to sudden, unforeseeable failures at the surface with practically no warning, if the tunnel face is not properly supported or if uncontrolled over-excavation is allowed. The authors emphasize also the fact that water table is located 10÷25 m above the tunnel, and roughly follows the shape of the surface topography. A large number of old wells and „minas” (old and small handmade water tunnels) are present in the area, with no historical information available. The vast number of these features has modified the hydro-geological characteristics of the ground, so that the groundwater moves not only in the porous medium and fractures, but also along the preferential channels represented by the „minas”, which strongly influence the underground water circulation. These hydro-geologic conditions have obvious implications in the Criteria for Selection of TBM (Table 6).

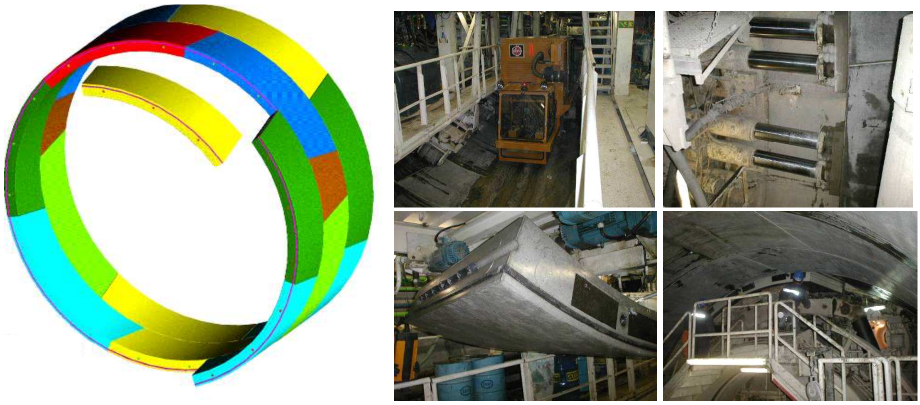

Fig. 11. Details of the lining for the EPB tunnel

Table 6. Criteria for Selection of TBM

	Rock Strength					Rock Structure							Water Ingress	
	Unilat. compr. MPa	cohesion Cu KN/m²	Face Support	Shield	Lining Install.	Jointing		Grain size		Face Support	Shield	Lining Install.	Volume per 30m³	Consequences on
						RQD	Distance	< 0.02 mm	< 0.06 mm					
Rock	> 250	-	-	-	-	100 - 90 %	>2 m			-	-		unlimited	pump capacity
	250 - 100		-		behind TBM	90 - 75 %	2.0 - 0.6 m			-	-	behind TBM	unlimited	pump capacity
	100 - 50		-		behind TBM	75 - 50 %	0.6 - 0.2 m			-	possible	behind TBM	> 20 l/s	pump capacity
	50 - 25		-	recom.	behind TBM	50 - 25 %	0.2 - 0.06 m			possibly mech.	recom.	in TBM area	> 10 l/s	face support
	25 - 5		-	recom.		< 25 %	< 0.06 m			mech.	required	under shield	> 5 l/s	face support
	5 - 1		recom.	required	under shield	< 25 %	< 0.06 m			mech., possibly EPB/ Slurry	required	under shield	> 2 l/s	methodology
Soil	< 1	>30	-	required	under shield	< 25 %	< 0.06 m	vary		mech., possibly EPB/ Slurry	required	under shield	> 2 l/s	methodology
		30 - 10	recom.	required	under shield			> 30 %		mech., possibly EPB/ Slurry	required	under shield	> 2 l/s	methodology
		10 - 5	recom.	required	under shield immed. grout.			> 20 %	< 50 %	EPB / Slurry	required	under shield immed. grout.	> 2 l/s	methodology
		5 - 1	required	required	under shield immed. grout.			> 10 %	< 30 %	EPB / Slurry	required	under shield immed. grout.	> 2 l/s	methodology
		0	required	required	under shield immed. grout.			> 10 %	< 20 %	EPB / Slurry	required	under shield immed. grout.	> 2 l/s	methodology

Due to the extreme variability and unpredictability of the geological conditions and surface constraints, the EPBs should be only been operated in closed mode. When excavating with an EPB in urban environment, the main concern was to reduce, as much as possible, the face and volume loss ahead, above and behind the TBM; so that settlements are maintained within acceptable limits. The design should therefore address the definition of the correct TBM working parameters to minimize face and volume loss; estimate of the extent and shape of the settlement trough; evaluation of the acceptable deformation limits of the buildings; definition of both preventive and remedial measures. As referred by Gaj et al. (2003), to deal with all these subjects in detail, the design package PAT (Plan for Advance of the TBM) was related to short tunnel sections ranging from 200 m to 1 km, and included the following separate documents: report on the geological investigation and its interpretation; report on building risk assessment; report and drawings about the monitoring of underground structures and surface buildings; report on the evaluation of the TBM working parameters; geotechnical profile with indication of the TBM working parameters; summary report on the Plan for Advance of TBM.

Towards the end of each section, the experience gained was summarized in specific back-analysis documents that helped optimizing each section. Thus, a process of continuous enhancement was implemented, with emphasis on the following (Guglielmetti et al., 2003; Gaj et al., 2003):

– in addition to „traditional" design information, such as geological evaluation and structural calculations, a specific set of TBM working parameters was delivered.
– in particular, the „report on the evaluation of the TBM working parameters" contained the definition of the reference value and the relevant operational range for: face support pressure; apparent density of the muck in the chamber; weight to be extracted at each ring; longitudinal grouting pressure and volume; additional bentonite slurry injection volume and pressure.
– the TBM working parameters were summarized from the PAT and delivered to the TBM crew in a simple form, called „Excavation Sheet".
– following the real-time and back-analysis activity, the Excavation Sheets were continuously updated, so that the PAT could be regarded as a „live document".

The implementation of the PAT and its continuous update has proved to be a very effective tool, as the geological conditions and the design parameters for the TBM are given in advance together with the instrumentation and the monitoring requirements (scheme in Fig. 12).

4.2 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, Guglielmetti et al. 2003). 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 (Fig. 13). Muck is removed 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. These comprise six segments and a key and dowel connectors in the radial joints, with guidance rods in the longitudinal joints.

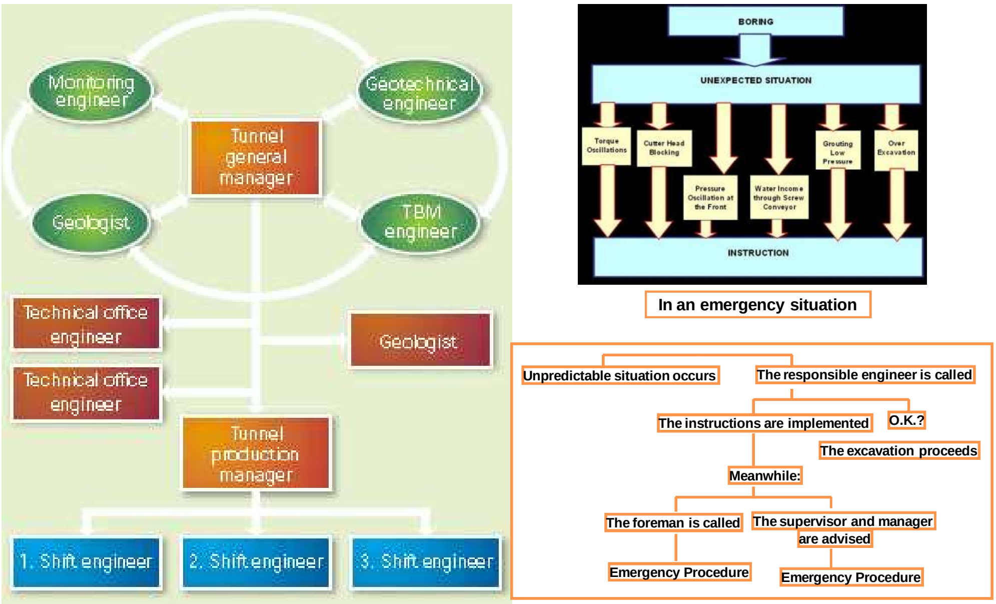

Fig. 12. Injection Tunnel team organization chart, where the contractor and designer representatives are inserted in elliptic and rectangular frames, respectively; reaction procedures (Gaj et al., 2003)

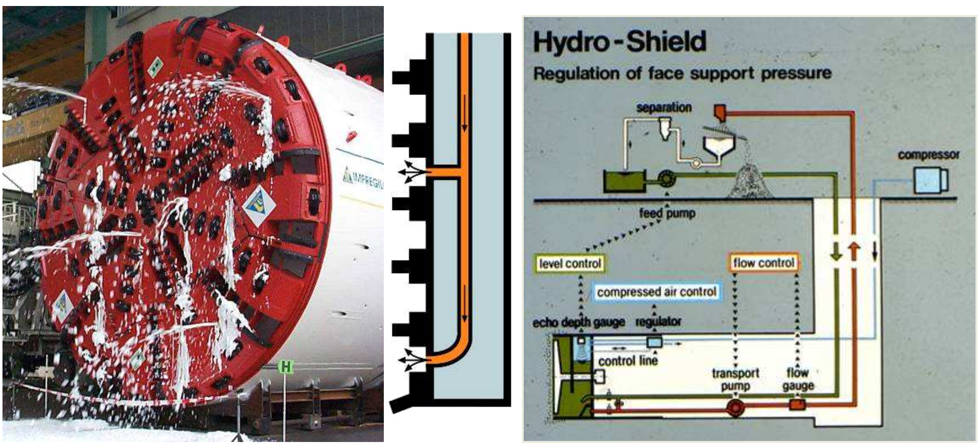

Fig. 13. Injection of conditioning additives into tool gap and pressurized slurry

The features of the EPB TBM are illustrated in Figure 14a. In a review paper by Della Valle (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 (system in Fig. 14b). In those papers issues proposed by the authors of paper are described and discussed.

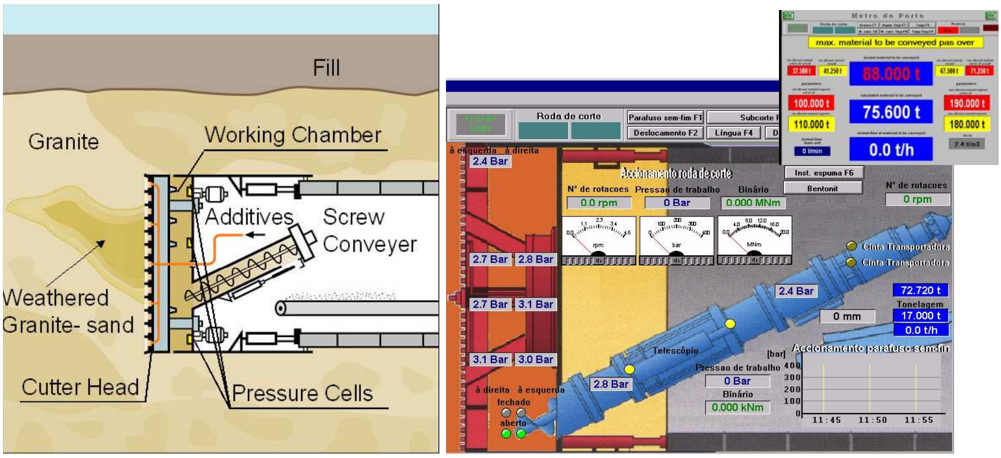

Fig. 14. Herrenknecht EPB TBM used in Porto (Babendererde et al., 2004): a) general characteristics; b) of the monitoring of the EPB machine with control of excavated material (weight)

4.3 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 (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 open mode. However, because of the unpredictable changes in the geological conditions described above, the panel of experts considered that the risk of operating in 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 the panel 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 pressurized mode than in an open or partially open mode.

4.4 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 15. 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 (Fig. 15).

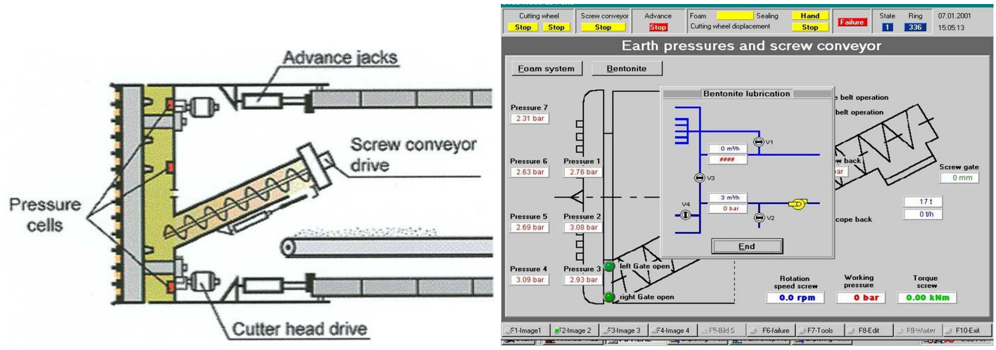

Fig. 15. Measurement devices for face support pressure (Babendererde et al., 2004)

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, with 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 Fig. 16 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.

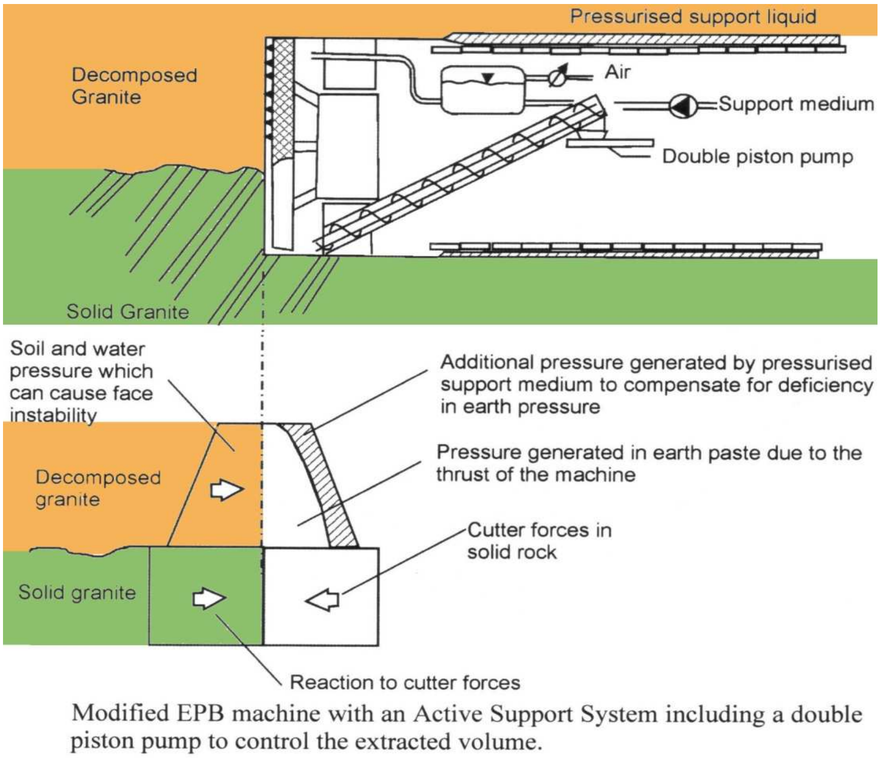

Fig. 16. Face support pressures in mixed face conditions in Oporto granite. An Active Support System for overcoming the support pressure deficiency is also illustrated

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. The deficiency of face support pressure can be compensated by the addition of an Active Support System, proposed by Dr Siegmund Babendererde (Fig. 16).

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.

So, resuming these were the modifications that were made to the machine configuration, leading to a higher degree of safety and better overall performance: (i) set up of an active Secondary Face Support System (SFSS); (ii) an automatic system that pumps bentonite slurry into the excavation chamber whenever the pressure drops below a preset level; (iii) installation of an Emergency Double Piston Pump (EDDP) after the screw conveyor, in order to deal with the liquid muck and uncontrollable support pressure oscillations; (iv) installation of a second belt scale, in order to cross-check the results of the first scale. In addition to the above, a certain number of significant adjustments have been made to settings of the software used to drive the TBM. In particular, the following three automatic alarm systems have been set up: one alarm for exceeding the extracted weight upper limit, which automatically stops the advance; one alarm for exceeding the face support pressure lower limit, which automatically switches on the SFSS; and one alarm for exceeding the lower limit of the apparent density of the muck.

4.5 TBM Performance

One of the key-factors for the successful completion of the project was the recourse to continuous analysis of TBM performance (real-time and back-analysis) as an important working tool. The Porto Metro Project represented, eventually, the first example of its application on a large-scale tunnelling project. As described by Gaj et al. (2003), the choice of this procedure was based on two factors:

– the assumption that the TBM advance was the potential cause of disturbance to the surrounding environment, and the settlements were the ultimate effect of the TBM advance, both the contractor and the designer agreed to integrate the conventional monitoring system – focused „only" on the effects of the tunnelling, i.e. ground and building movements – with a new system, which would analyze the causes of such movements.
– based on the different type of ground (soil-like or rock-like) and the overburden, the delay between the passage of the TBM (cause) and the occurrence of the settlements (effect) could vary from a few hours to several days or even weeks.

By analyzing the TBM data, it was in many cases possible to know in advance the potential of induced settlements before they could reach the surface (or even before being recorded by the monitoring system). As an important consequence, Gaj et al. (2003) describe that it was possible to apply counter-measures in a timely manner. In fact, with the main scope of anticipating the potentially negative impact of tunnelling at the surface (settlements or even collapses), the TBM engineer was fully dedicated to the analysis of the TBM behaviour, under distinct levels.

– Real-time analysis: verification of incorrect operation of the TBM and consistency with those foreseen; evaluation of TBM operation; checking the main parameters that can influence the surrounding ground and the surface (i.e. extracted weights, face support pressures, backfill grouting volumes, etc.), addressing risk and advise on necessary counter-measures;
– Back-analysis: summary of the vast volume of data collected in each section of tunnel and, based on the global analysis, advice on any optimization for the following section.

In the end, one of the main challenges on the project was to manage the advance through mixed face conditions, accomplishing two different goals: excavating very hard and abrasive granite with satisfactory advance rates; and supporting very weak and loose material resulting from the weathering of the granite. Being these two objectives almost contradictory, the analysis of the TBM operation needed to include a wide range of parameters, some of which related to the control of face and void stability, and others to the excavation process itself (schemes in Fig. 17).

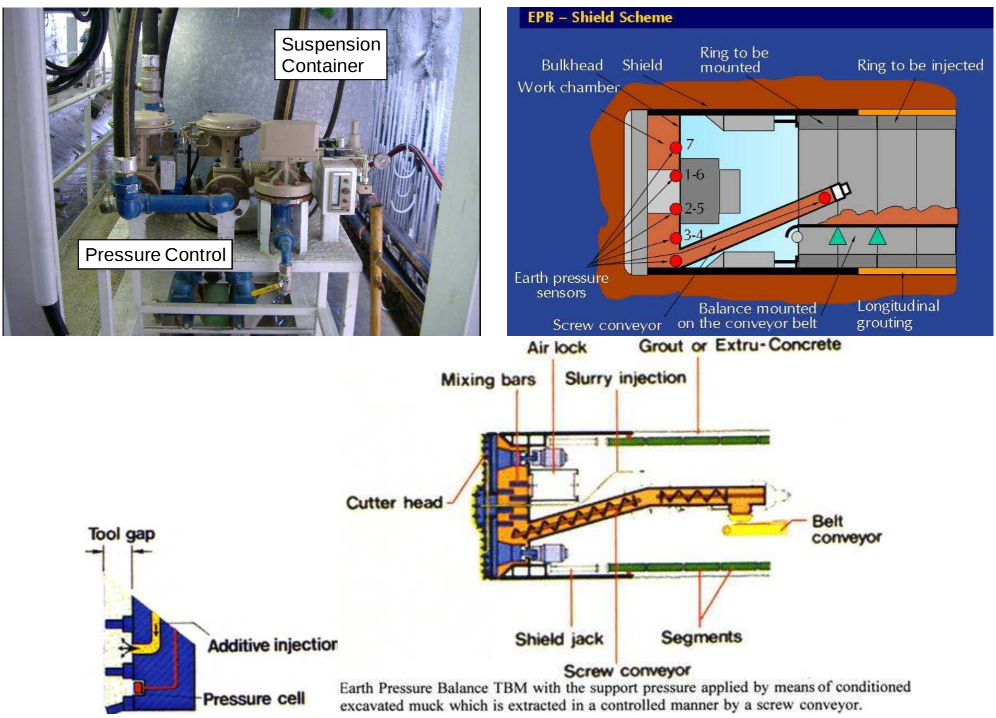

Fig. 17. The solution for Metro do Porto TBM – EPBM. Secondary Face Support System (SFSS). Earth Pressure Balance TBM with support pressure applied by means of conditioned excavated muck, extracted in a controlled manner by a screw conveyor. Detail of tool gap and working chamber with pressure cell

For each TBM parameter, the PAT defined an operational range and counter-measures to be applied if the attention/alarm limits were exceeded and was done the verification that fundamental TBM parameters complied with the specifications defined in the PAT (Gaj et al., 2003). At the second stage, as a natural evolution, other operational parameters were included in the study, including: penetration (advance) rate; total thrust and cutterhead thrust; cutterhead torque; cutting tools wear rate; conditioning agents (foam & polymer) consumption. For these parameters no limits were imposed in the design, but their trends were carefully analyzed, as they could give useful information about the ground conditions and correct operation of the machine itself.

4.6 Control of the Extracted Muck Weight

The TBM incorporated a conveyor belt system for the removal of excavated material; hence the measurement of the extracted muck weight was carried out by using two belt scales. As described by Gaj et al. (2003), thanks to the ability to cross-check the figures recorded on the scales, the system has proved to be generally reliable, with an accuracy of 3÷4%.

In order to adjust the reference value and the operational range as much as possible, to fit the actual geological conditions, it was necessary to execute frequent face mapping and estimate the ground in-situ density, at least every 15 m. The figures recorded on the scales had to be considered as gross weights and before comparing them with the reference value it was necessary to subtract the weight of the conditioning agents added during the excavation (average 20÷30 m^3 of water with polymer – Figure 18). Some fluctuations around the reference value are unavoidable, as the extracted muck weights vary according to the compaction grade of the material inside the excavation chamber, which in turn is highly variable at each ring. The tunnel alignment included some very tight curves (R=200 m); in these curves, due to the misalignment of the backup gantries and the bending of the conveyor belt itself, the balances were not correctly loaded and the recorded weight figures less accurate (Gaj et al., 2003).

4.7 Management of Face Support Pressure

The management of face support pressure is determinant for guaranteeing the successful operation of an EPB machine (Gaj et al., 2003). The design approach requires the application of a minimum effective pressure. In simple terms, it is required that the support pressure be transferred from the bulkhead to the face via solid particles and not only through the pore pressure or compressed air (Fig. 18). The operational consequence is that the chamber must always be full of solid matter with an adequate density, defined at approximately 14k N/m^3 (Gaj et al., 2003).

As a safety measure, the reference support pressure was defined and measured at the top of the bulkhead (1 m below the tunnel crown) so that the other pressures were higher (Gaj et al., 2003). The density of the muck was evaluated by measuring pressures at different bulkhead heights and dividing the difference in pressure by the corresponding difference in height. As described by the referred authors, due to the heterogeneity of the material in the chamber, and the dynamic effect caused by both the rotation of the cutterhead and the compression of the bulkhead, the instantaneous density figures fluctuated significantly. Therefore, the density had to be considered „apparent”.

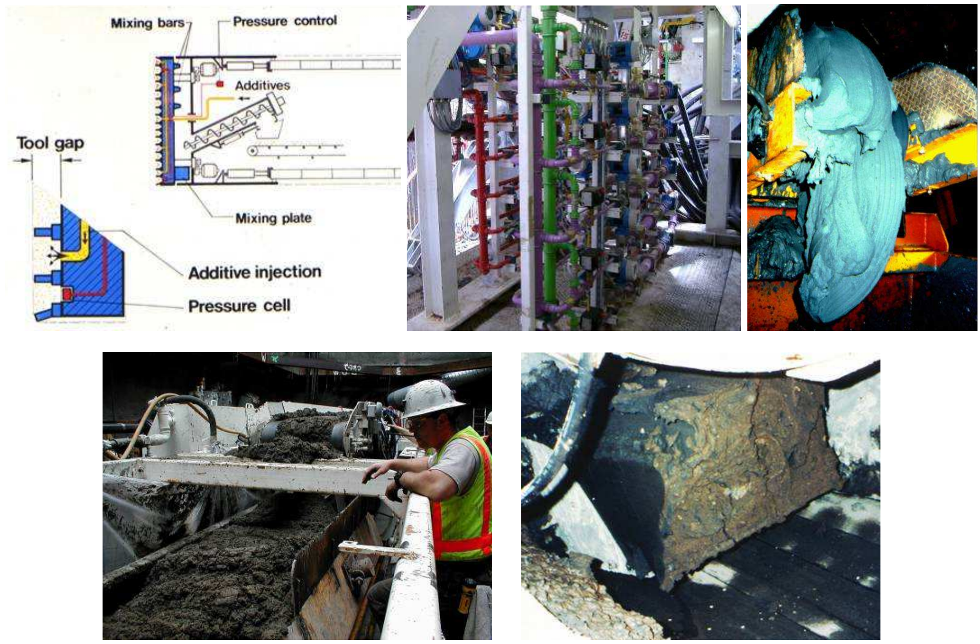

Fig. 18. Conditioning: injection of additives, foam generator, natural ground (clay as support medium) and Earth Paste Conditioned with Foam (Proper Conditioning and Earth Paste when air diffused after TBM stop)

Figure 19 presents some illustrations of the management of face support pressure. In the granite of Porto, both polymer and foam proved to work successfully as soil conditioning agents. The polymers were more reliable, even in delicate conditions, but they cannot help to reduce the very high tool-wear rate, due to the high abrasivity of the granite, which was one of the main problems. The use of foam has instead helped to greatly reduce the wear rate, however, required more complex settings and smoother operation of the screw conveyor (Fig. 20).

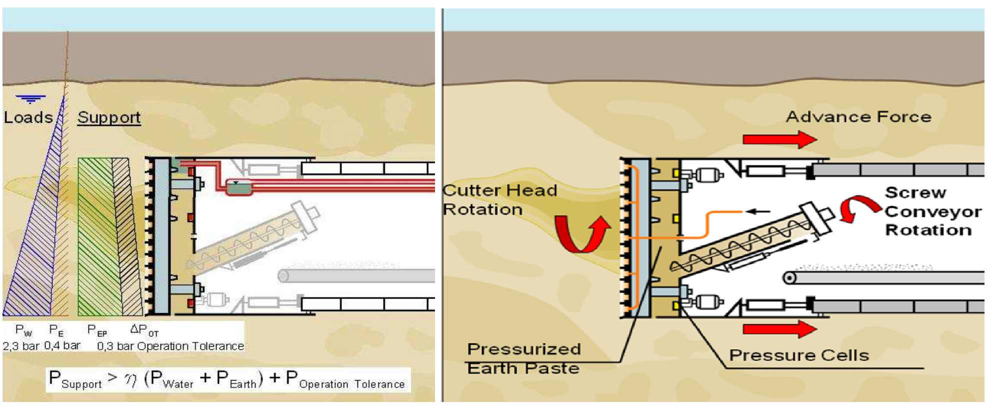

Fig. 19. Management of face support pressure: measuring pressures at different bulkhead heights (dividing difference in pressure by the height between monitoring points) and earth pressures sensors

Fig. 20. Cutterhead tool wear and its measurement

4.8 Control of Primary and Secondary Grouting

Backfilling around the segmental lining was carried out using a longitudinal grouting system composed of six injection lines, evenly distributed around the tail skin (Gaj et al. 2003). To check the efficiency of the primary grouting, as a standard practice, grout cores were extracted every ten rings and, if necessary, secondary radial injections were carried out at pressures of up to 4 bar. In critical areas, i.e., under sensitive buildings with low cover, the conical void around the machine was filled with bentonite slurry, continuously injected through ports located along the shield. This provided additional help to reduce volume loss due to the TBMs' passage (Gaj et al. 2003).

4.9 Face Conditions and TBM Parameters

Analysis of the TBM operational parameters allows detection of the ground conditions, as traditionally seen in oil drilling or geotechnical investigation campaigns. In particular, the combination of cutterhead thrust, advance speed and disc wear gives a clear idea of the face conditions, e.g. the harder the face, the higher the cutterhead thrust and disc consumption; the harder the face, the lower the advance speed. Thanks to frequent face surveys, it was possible to accurately calibrate the interpretation of the trend of these parameters and then, whenever a direct survey of the face was not carried out, it was still possible to recognize the ground conditions through which the TBM was advancing (Table 7).

Table 7. Matrix of face conditions and TBM parameters

Face conditions versus cutterhead thrust		**Average cutter-head thrust (kN)**					
		4000	**6000**	**8000**	**10000**	**12000**	**14000**
Geomechanical group	g6						
	g5						
	g4						
	g3						
	g2						

4.10 TBM Production and Utilization Rates

Even taking into account the high difficulties of these geological and geotechnical environment, it was to achieve very good results in terms of advance rate and production.

The TBMs were normally operated on a 24hr basis, working a six-day week. Almost one shift per day was used to execute cutterhead maintenance under hyperbaric conditions.

The following main results have been achieved:

– average daily production: 5 rings = 7 m;
– best day (06/02/2002): 13 rings = 18.2 m;
– best week (7-13 October 2002): 56 rings= 78.4 m;
– best month (August 2003): 180 rings = 252 m.

The site management and level of skill of the workers is shown in the very high TBM work rates.

Considering that the two TBMs have been operated in a very sensitive urban environment – always in closed mode - these results can be regarded as very positive and above the usual standard (Gaj et al. 2003).

The final breakthroughs of the TBM machines on the completion of the drive from Campanhã to Trindade and from Salgueiros to Trindade are illustrated in Fig. 21.

a) b)

Fig. 21. Final breakthrough of the TBM machines: a) on the completion of the drive from Campanhã to Trindade (12 Oct. 2002); b) on the completion of the drive from Salgueiros to Trindade on 16 Oct. 2003

5. 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 (Babendererde et al. 2004), 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 developed when the project was planned.

6. Acknowledgements

The authors wish to acknowledge the permission of Metro do Porto to publish the details contained in this paper. The cooperation of Transmetro during the construction is also acknowledged. Part of this paper was adapted from two papers that have thoroughly described the tunnelling construction with TBM (Gaj et al., 2003 and Babendererde et al., 2004), with some transcription of descriptions and figures, for which the authors wish also to make specific recognition to the responsible for those comprehensive papers.

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