EVALUATING THE REQUIRED FACE-SUPPORT PRESSURE IN EPBS ADVANCE MODE

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1. INTRODUCTION

The stability of the face is one of the most important factors in selecting the adequate method of excavation of a tunnel. This is particularly true for mechanised tunnelling and specific boring machines (TBM), as, for examples, the Earth Pressure Balanced (EPBS) shield and the Slurry Shield, have been developed in the recent decades for managing the instability of the excavation profile in unfavourable geotechnical and hydrogeological conditions, with challenge external constraints.

As a logical consequence, the evaluation of the stabilizing face pressure is a critical node for both the design and the construction phase. In spite of the importance of the subject, specific recommendations or technical norms are not known as common guidance for the design, and, in the current practice, different approaches are often employed both to evaluate the stability condition of the face and to assess the required stabilizing pressure.

The present note deals with these aspects and gives a contribution on the basis of theoretical and experimental considerations, with particular reference to the realization by EPBS of the Porto Light Metro in Portugal (Guglielmetti et al., 2002; Grasso et al., 2002).

In the first part of the paper (§2), some referenced methods for evaluating the stability of face are presented and are compared using practical examples. Then (§3), the consequent problem of defining the adequate design face pressure is treated through analysing international practice and experimental research in laboratory (AFTES, 2001). Finally, the principal results of this study are evaluated on the basis of mentioned experience of the Porto Metro, deriving practical indications for establishing a correct design approach.

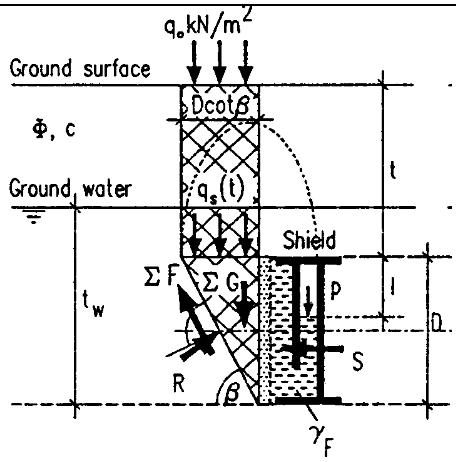
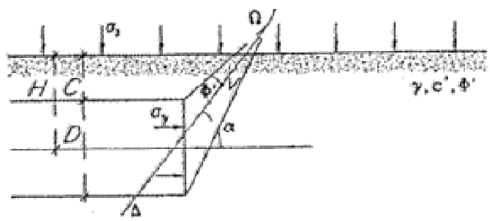
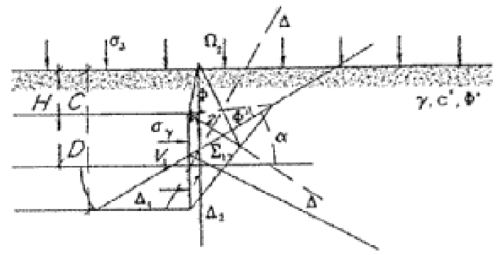
2. ANALYSIS OF THE STABILITY OF THE FACE

2.1 Limit equilibrium methods

The study of the stability of the tunnel face is a complex problem and a very detailed solution can be developed only on the basis of three-dimensional numerical analysis. However, in many cases also the reference to the so-called methods of limit equilibrium (LEM) gives satisfactory solutions, representing still an important practical tool for design, especially when based on tri-dimensional failure models. A comprehensive treatment of these subject is furnished by W. Broere (2001). In table 2.1, some referenced LEM, applicable to the general condition of cohesive-frictional ground, are shown.

The static stability of the face could not be a sufficient design criterion for avoiding settlement on the surface: particularly for shallow tunnels in urban environment, it is also necessary the stability of the excavation both around the shield and the lining (Aristaghes & Auturi, 2003), as well as to preserve the hydrogeological conditions. For this reason, in the practical applications of limit equilibrium methods in section 2.2, the equalizing water pressure in the working chamber of EPBS is always highlighted.

Tab.2.1: Selected methods for analysis of face stability in cohesive-frictional ground

Method and Basic formulations	Scheme
<u>1. Method of Jancsecz and Steiner (J&S, 1994)</u> According to the model of Horn (1961), the three dimensional failure scheme consists of a soil wedge (lower part) and a soil silo (upper part). The vertical pressure resulting from silo and acting on soil wedge is calculated according the Terzaghi's solution. A three dimensional earth coefficient ka_3 is defined as: $ka_3=(\sin\beta\cos\beta-\cos^2\beta\tan\phi-K\alpha\cos\beta\tan\phi/1.5)/$ $(\sin\beta\cos\beta+\sin^2\beta\tan\phi)$ where: $K\approx[1-\sin\phi+\tan^2(45+\phi/2)]/2$ $\alpha=(1+3t/D)/(1+2t/D)$	
<u>2. Method of Leca & Dormieux</u> (L&D, 1990) This method is based on the upper and lower limit theorems with a 3D-modelling. The upper($^+$) and lower ($^-$) limit solutions are derived by means of cinematic and static method, respectively, so giving an optimistic (by defect) and pessimistic (by excess) estimation of the stabilizing pressure. In the case of dry condition, the stabilizing face pressure σ_T is equal to (Ribacchi, 1994): $\sigma_T=-c'*ctg\varphi'+Q\gamma*\gamma*D/2+Qs*(\sigma_S+c'*ctg\varphi')$ $Q\gamma$,Qs = adimensional factors (from normograms), function of H/a and φ', where: a = radius of the tunnel; H= thickness of the ground above the tunnel axis.	  Note: a third failure mechanism refers to blow-out failure in very shallow tunnel (σ_T is so great that soil is heaved in front of the shield).

3. Method of Anognostou and Kovari (A&K,1996)

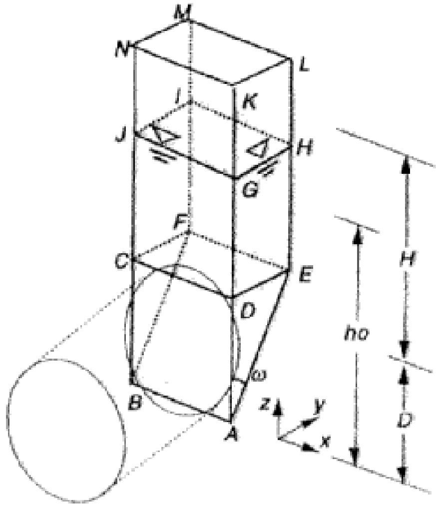
This method is based upon the silo theory (Janssen, 1895) according to the tri-dimensional model of sliding mechanism proposed by Horn. The analysis is performed in drained condition, and a distinction between the stabilizing water pressure and effective pressure in chamber of EPBS is presented. If a gradient between water pressure in the chamber and in the ground exists, destabilizing seepage forces occur and a higher effective pressure is required at the face. However, accepting this flow, the total stabilizing pressure is lower than in the case of imposed hydrogeological balance.

The effective stabilizing pressure (σ') is:

$\sigma' = F_0\gamma' D - F_1 c' + F_2\ \gamma'\Delta h - F_3 c'\Delta h/D$

If the material in the chamber is in a fluid state, $\sigma'=0$ and solving the equation for Δh the necessary water pressure for equilibrium is derived.

F_0, F_1, F_2, F_3= adimensional factors from normograms, function of H/D and φ'.



Note: The original analysis consider k_o=0.8-0.4 for the prism and for the wedge (tunnel level), respectively.

2.2 Practical application of the methods

A practical application of these methods refers to the design and construction follow-up of the new light metro of the city of Oporto (Portugal), currently under construction by EPBS. As an example, a section with the features reported in table 2.2 is analysed (reference to the above scheme of A&K method) and the results are summarised in Table 2.3.

Tab.2.2: Main features of the examined section

Geology	Complex conditions: prevalent completely weathered granite (W5) and/or residual soil (W6), with local presence of boulders of relatively less weathered granite (W3/W4).
Geotechnical condition	γ'=10-12kN/m^3; c'=0-20kPa; φ'=30-34°; k_o=0.5(assumed); K=10^{-5}-10^{-7}m/s
Contour conditions	H=18.2m; h_o=14.8m; D=8.7m

Note: K=coefficient of permeability

Before comparing the results of Table 2.3, the following should be noted:

- taking into account the residual uncertainty, the analyses have been performed considering both a "mean" (mean shear parameters in tab.2.2) and a "worst" scenario (lowest values): note that the latter coincides with the design approach;
- A&K analyses are performed supposing an hydraulic equilibrium, so that water flow and seepage forces do not occur. In other words the groundwater pressure is completely compensated by the fluid pressure in

the working chamber. Hence $\Delta h=0$ and the water table is not modified, avoiding the risk of settlement on the surface;

- the method of L&D refers to dry condition: an approximated solution is proposed arranging the original formula in terms of effective parameters and adding the groundwater pressure. According to this approach a weighted value of the unit weight of the ground ($f(\gamma',\gamma)$) is considered in the equation of equilibrium;
- the results are directly expressed in terms of the required stabilizing pressure: in particular, if the required σ_T is higher than groundwater pressure it means that the tunnel face is unstable even in the condition of hydrogeological balance.

Tab.2.3: Results of the analysis of stability (mean→**worst** scenario; at the tunnel crown/at tunnel invert)

Method	Notes	$\sigma_{T(dry)}$ (kPa)	σ_T (kPa)	σ'_T (kPa)
1.(J&S)			(115/221)→**(130/238)**	54/73→**69/90**
2.(L&D)	Upper limit ($^+$)	2→**20**	(61/148)→**(74/161)**	(0→**13**)
	Lower limit ($^-$)*	10→**30**	(62/149)→**(81/168)**	(1→**20**)
3.(A&K)			(61/148)→(**82/169**)	0→**21**

Note: *Ribacchi (1994); the values in parenthesis are extrapolated for comparison, taking into account that the existing groundwater pressure is 61/148kPa.

According to the results in the table, it is possible to observe that:

- for the worst scenario, all the analyses confirm the instability of the face and the consequent necessity of a confinement; if the mean values are used, only for J&S method the hydrogeological balance is not sufficient for stabilizing the face;
- there is a practical coincidence between the results of L&D (lower limit) and A&K methods; as derivable from the paper of the latter Authors, a more significant difference among the two methods should be expected for both higher values of cohesion and larger diameter of excavation. These results are also in good agreement with experimental findings from centrifugal models (Chambon e Cortè, 1994);
- according to the results from the L&D methods, and in particular to the reduced difference between the upper and the lower limit analysis, the solution appears well defined;
- the confining pressures obtained with J&S are higher than the other methods and, as in some way expectable, intermediate between them and the active effective pressure σ'_{Ka} (87/117→**107/139**kPa).

3.DEFINITION OF THE DESIGN FACE PRESSURE

In the previous section, the practical application of different approaches verified the instability of the tunnel face at the examined section. This implies the necessity of an adequate confinement and, more in general, of a suitable tunneling method: in the cited example a fully mechanized excavation by means of an EPBS.

The EPBS principle involves a cutting wheel operating in front of a chamber entirely filled with excavated soil. Material is extracted in a controlled manner from the excavation chamber using a screw conveyor, which governs the pressure of the excavated soils and provides earth pressure balance to the excavated face. Face pressure is controlled by balancing the rate of the advance of the shield and the rate of discharge of the excavated material through the screw conveyor.

The equilibrium condition occurs when the conditioned ground in the working chamber reaches the maximum density for the acting pressure and the volume of the material extracted from the screw conveyor equalizes the theoretical one removed by the cutterhead (Maidl et al.,1996). In this condition the pressure released by cutting face of the EPBS should be equivalent to the earth pressure and the ground ahead of the cutterhead remains in elastic domain.

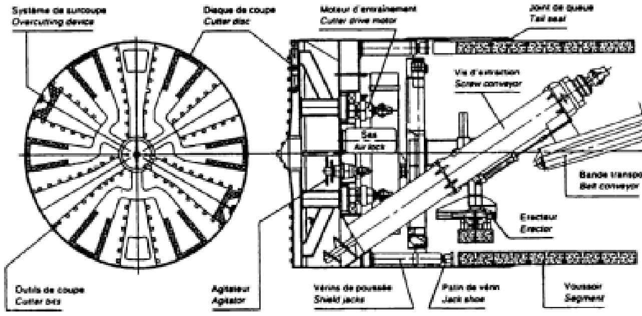
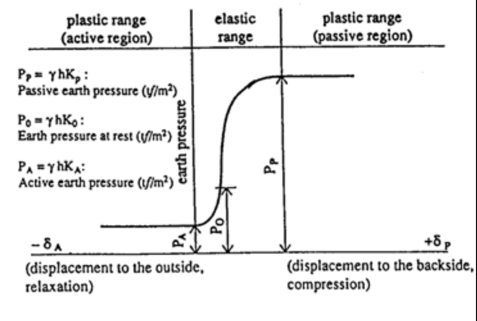


Fig. 3.1: A typical scheme of EPBS

For closed-type shield tunneling work, this is generally considered the optimum from the viewpoint of minimizing face deformation and keeping the face stable (Kanayasu et al.,1995, Miyazachi et al., 1984, as reported by Reda, 1994). Unfortunately, it is generally difficult to determine the coefficient of at-rest earth pressure.

An empirical rule for normal-consolidated soil (NC) was proposed by Jaky (1944) and the derived simplified form is: $k_o(NC) \cong 1-\sin\varphi'$. As reported by Lancellotta (1987), the following equation was proposed for over-consolidated soil (OC): $k_o(OC)=k_o(NC)*OCR^{\alpha}$, where OCR is the Over Consolidation Ratio and $\alpha=0.46\pm0.06$ for low-sensitive clays (Jamiolkowski et Al.,(1979).

Furthermore, it is common opinion (see for example Reda, 1994) that the stability of the excavation is controlled if the face support pressure is between the active and the at-rest ground pressure (i.e. $\sigma_{ka} < \sigma_T < \sigma_{ko}$). The earth pressure becomes active or passive when the ground deforms plastically towards the cutterhead or in the opposite direction (i.e. the ground is pushed by the EPBS), respectively (fig.3.2).



Outer Diameter (mm)	Soil Type	Face Pressure
7,450	soft silt	earth pressure at rest
8,210	sandy soil, cohesive soil	earth pressure at rest + water pressure + 0.2 kgf/cm²
5,540	fine sand	earth pressure at rest + water pressure + fluctuating pressure
4,930	sandy soil, cohesive soil	earth pressure at rest + (0.3 ~ 0.5 kgf/cm²)
2,480	gravel, bedrock, cohesive soil	earth pressure at rest + water pressure
7,780	gravel, cohesive soil	active earth pressure + water pressure
7,350	soft silt	earth pressure at rest + 0.1 kgf/cm²
5,860	soft cohesive soil	earth pressure at rest + 0.2 kgf/cm²

Fig.3.2: Relationship between earth pressure and displacement (left) and Examples of face pressure (EPBS shield) in Japan (Kanayasu et al.,1995)

Kanayasu et al., collaborators of a survey on Japanese Shield Tunneling, pointed out that in most cases the active earth pressure is used as the lowest permissible level of face pressure, but, more in general, there is currently no clear principle for defining the design face support pressure.

Examples of face pressure adopted by EPBS in Japan are summarised in Fig.3.2. On the basis of available information, it seems possible to observe that in the European practice the hydrostatic pressure (σ_W) is generally assured and a supplementary component for the ground thrust is added. A quoted rule of thumb (COB, 1996, in Broere, 2001) is $\sigma_T=k_a\sigma'_V+\sigma_W+20kPa$, but also $\sigma_T=\sigma_W+20kPa$ has also been followed on the basis of practical experiences (see for example Leblais et al., 1996).

We are describing different opinions for defining the face support pressure, but is it really fundamental the exact evaluation of it?

Useful contributions for a better understanding of the face confinement in EPBS mode derive from laboratory tests simulating in reduced scale the process of excavation. In the next section, a short insight is presented, referring in particular to the results of the research project "Eupalinos 2000" (AFTES, 2001). As we will see, these results contribute to move the focus of the problem.

3.1 Simulation in laboratory of the EPBS advance mode

Recently, a synthesis of results of the French national project "Eupalinos 2000" on "Mechanized excavation in heterogeneous ground" and "Earth Pressure Balance Shield" have been published by AFTES (2001). In particular, the theme B1 "Control of the confinement by earth pressure: Laboratory studies on reduced models" is of interest for the argument here analysed. On this topic, n. 11 specific reports (1998-2001) were presented showing the progress of the research.

In the Figs. 3.3 and 3.4, the model of EPBS used in laboratory (scale 1:10) and the different types of cutterhead employed are shown, respectively.

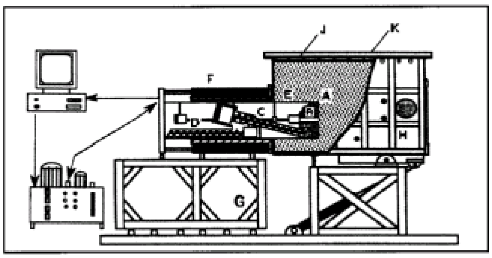


Fig.3.3: EPBS model used (AFTES,2001)

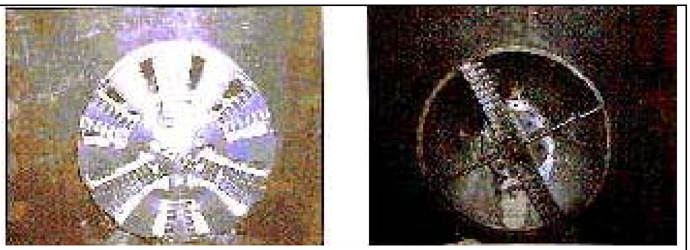


Fig.3.4: Different cutterheads employed (AFTES, 2001)

The tests simulated the excavation into incoherent dry soil (fine sands with $\varphi=33°$ and $\gamma_d=13\text{-}17kN/m^3$), continuously monitoring the pressure in working chamber of EPBS model and in the surrounding ground, as well as deformations and settlement on the surface.

Analysing this technical documentation, the following comments are derivable for the specific issue here examined (refer to the original tests for a complete and detailed discussion):

- Mainly two control parameters are able to govern the driving of the boring machine: (a) the ratio R between the mass of the actually extracted material from the screw conveyor and the theoretical one, and (b) the pressure in the ground to be excavated. If the latter seems not of practical importance (at the moment, it is difficult to imagine a continuous monitoring system in advance to the TBM), the former is the key parameter for the advancement;
- the ideal functioning regime is reached when R=1: this condition is the "Regime of equilibrium" and should be attained starting the excavation with the adequate confinement and avoiding the plasticization of the ground in advance (Fig.3.5a);
- when the material extracted is less than the theoretical one (R<1), the passive state crops up in the ground, plastic zones develops in advance up to few diameter and the pressure in the working chamber increases (Fig. 3.5b);
- in the opposite case, when extracted is more than theoretical (R>1), the ground enters in active state, large deformations occur nearly at the vertical to the cutterhead up to the surface and the pressure in the chamber decreases; it is important to observe that a tendentially constant pressure level in the chamber can be obtained if R is kept constant (Fig.3.5c);
- as derivable from the previous comments, the pressure in the chamber does not suffice to establish the actual regime of excavation (i.e. if over- or sub-extraction is currently occurring); in addition, it usually registers high fluctuations for the rotation of the boring wheel;

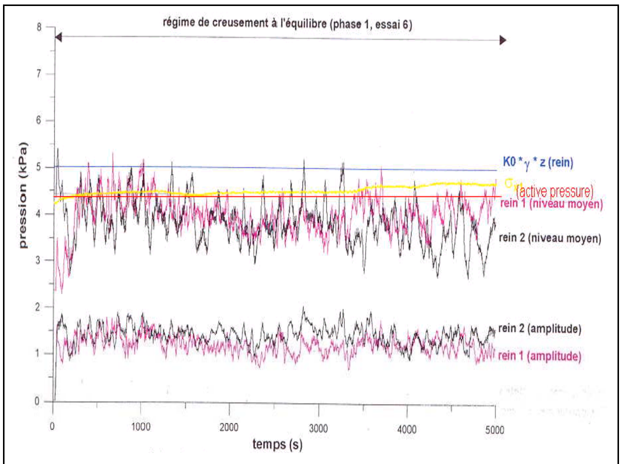


Fig.3.5a: Regime of equilibrium (R=1)

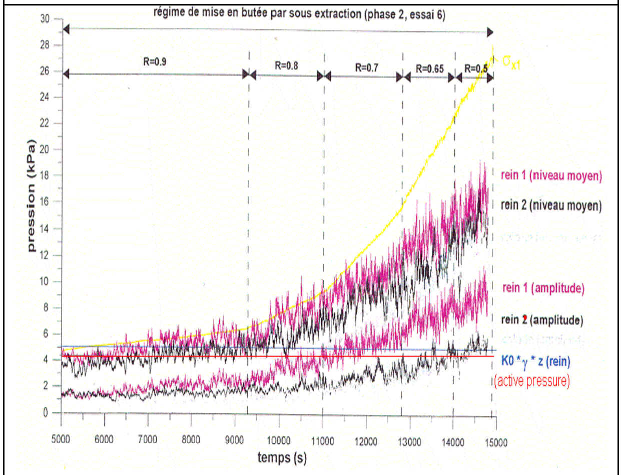


Fig.3.5b: Regime of sub-extraction (R<1)

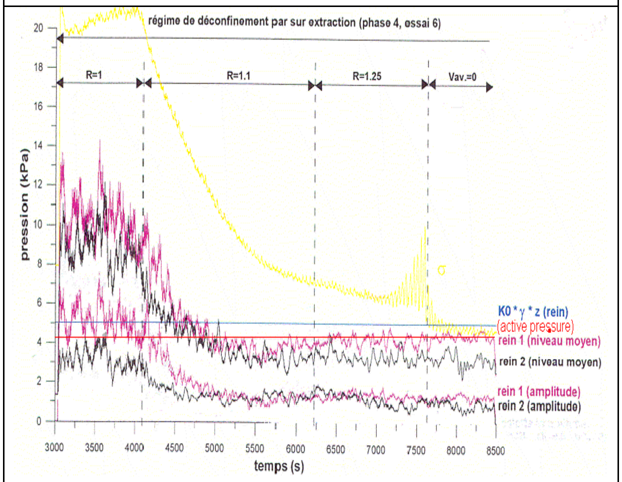


Fig.3.5c: Regime of over-extraction (R>1)

- however, experimental data show that when a regime of equilibrium is maintained and the pressure in the chamber is stable, its maximum values (peaks) are 0.9÷1.1 times the existing pressure of the ground at rest ($\gamma*H*k_o$). Moreover, analysing the graphs in Fig.3.5, we could add that the mean pressure values are approaching the active earth pressure ($\gamma*H*k_a$).

As described in the previous section, the laboratory experiments are confirming that the optimum regime of advancement (OAR), even in terms of controlling of displacements on the ground surface, involves: (1) balance of the extracted vs. removed material and (2) stable pressure condition in the working chamber. When these conditions are attained the pressure released by the cutting face of the EPBS should equalize the at-rest earth pressure. In other words, the choice of the face pressure could not be the primary design problem, as rather the goal to attain through the assessment of (1)&(2) conditions.

A question arises from the experience reported in Fig.3.2: when the recorded pressure values are different from the at-rest earth pressure, does it mean that:

1) the OAR was not been reached, or

2) the applied pressure was in reality equalising the "true" at-rest earth pressure (or, at least, the pressure required for maintaining the ground in elastic domain)?

In fact, it is important to observe that "true" geotechnical parameters must be theoretically considered in evaluating the equilibrium earth pressure, i.e., in other words, the factor of safety commonly used for design should be neglected. If the applied pressure is significantly higher than necessary (due to the safety factor), it must be accepted that the described OAR (i.e. R=1 & constant pressure in the chamber) cannot be maintained.

3.2 Additional comments for the real excavation process

The application in practice of the described theoretical and experimental results may be limited by an objective difficulty to verify the weight equilibrium condition. In fact, the verification involves these important aspects:

- the exact in-situ density of the ground is often unknown, especially in complex geotechnical environment and only an approximated value can be estimated: as a consequence the definition of the weight to be excavated could not be precise as required;
- the muck is frequently conditioned with additives (foams, polymers, bentonites, etc.) for improving its granulometry and workability: then, the weight and, more in general, the effects of these additives must be considered (see, for example, Herrenknecht and Maidl, 1995);
- according to some Authors (see, for example, Reda, 1994) the existence of pressure gradients in the working chamber can determine significant difference among the true applied confinement at the face and the measured pressure on the bulkhead. In the same direction of the described results of laboratory tests, for this reason, Maidl and Cordes (2003) derive that the control of the confinement pressure provides not guarantee for a stable tunnel face.

On the other hand, as we will see in the following, the tendential final pressure in the chamber during "standstill" seems to be a good indicator of the required stabilizing pressure.

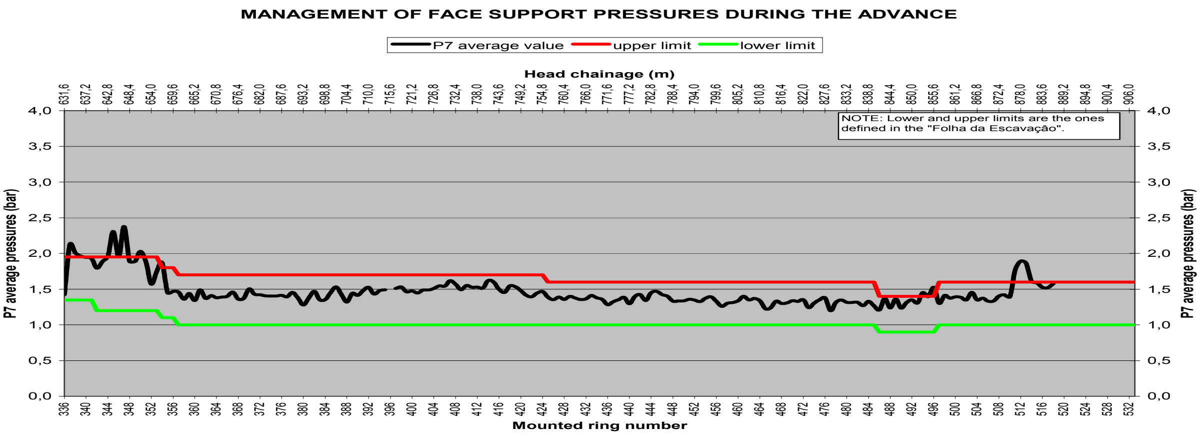
3.3 Example of practical experience

As the Eupalinos 2000 project pointed out, during the excavation it is important to avoid failure process by applying an adequate confinement to the face and to drive the EPBS for achieving and maintaining the optimum regime of advancement (OAR). This operation is essentially performed by means of an appropriated regulation of both the advance rate and the screw conveyor speed.

The described approach was actually applied for managing the advancement of the EPBS of the Porto Metro (for a complete description of the innovative construction techniques applied, see Guglielmetti et al., 2002).

In this project the working range of the main excavation parameters is defined in the design and then the same parameters are continuously controlled during the excavation process.

An example of this construction management process is shown in Fig. 3.6, where the actual fluctuations of the face support pressures and excavated weights within the working range are represented.



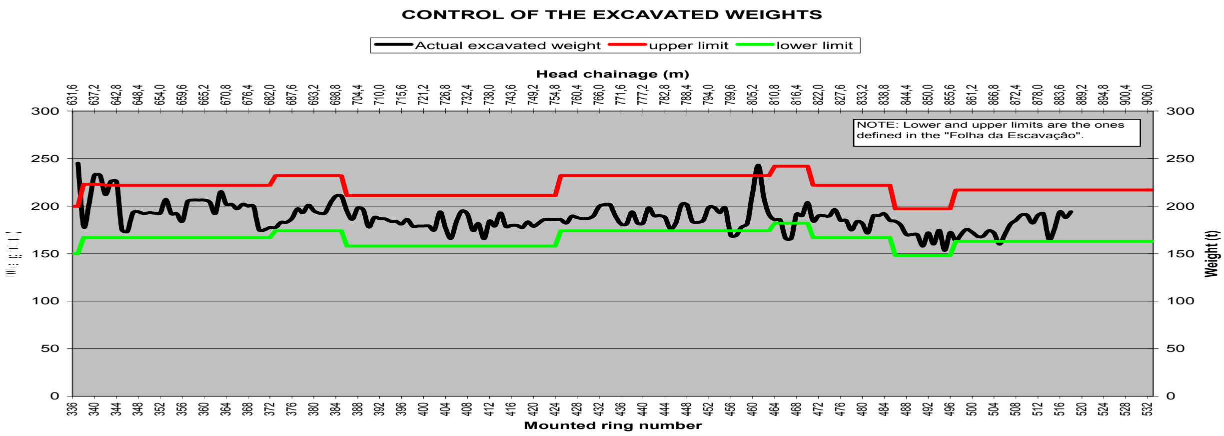


Fig.3.6: Example of control of bulkhead (P7) pressure and excavated weights during the EPBS advance (Porto Metro).

The following comments are necessary:

- the measurements of pressure are referred to the sensor "P7" on the bulkhead; to be noted that n.7 sensors (P1→P7) are present and that the P7 is located at ~1m below the tunnel crown;
- in the examined section, the confinement of the face is basically calculated according A&K method: then, according to Tab.2.3, the confinement of the face requires the hydrogeological balance ($\Delta h=0$, $\sigma_W=61/148$kPa) and the effective pressure $\sigma'_T = 21$kPa. It is important to observe that the latter component imposes a sufficient density of the muck in the working chamber to transfer the "grain to grain" contact pressure (in this case, $\gamma_{muck}>14kN/m^3$ was fixed). At P7 level the required total pressure is $\sigma_{T(P7)}=(61+10+21)=92$kPa;
- in favour of safety, an additional pressure of 20kPa is imposed to intercept also the pressure fluctuations in the chamber and, also considering the adjacent sections, a final $\sigma_{T(P7)}=120$kPa is fixed as design reference, with 100-160kPa as relative lower and upper alarm limits, respectively. The resulting design pressure is also nearly coincident with the value obtained by means of J&S method (mean scenario).

In Fig.3.7, both the pressure recorded on the bulkhead (P7) and the difference between the theoretical and the actual flow of material through the screw conveyor are shown. The data have been collected every 10 seconds, during the excavation for the examined ring (1.4m long) and its positioning (final part of the graphs, "standstill"). The following observations are possible:

- the bulkhead (P7) pressure does not appear constant and a "sinusoidal" type curve is traced;
- this behaviour seems to be mainly determined by the EPBS operator, by means of a continuous adjustment of the screw conveyor speed, in order to achieve the objective of maintaining both the pressure within the design limits and the balance of weight in equilibrium;
- however, due to obvious safety reasons, the pilot tends to operate the screw conveyor as slow as possible and therefore a general tendency of under-extraction is observed, thus forcing the pressure to increase;
- nevertheless, when the operator tries to limit the excessive growth of pressure, by increasing the flow of material through the screw conveyor (and then moving towards a regime of equilibrium (R→1) or over-extraction), a stable condition is anyway not reached and the pressure quickly decreases;
- finally, the reduction of pressure beyond the design lower limit is avoided by decreasing the screw conveyor speed and then an analogous cycle starts again;
- the "natural" tendency of the pressure to reduce is displayed also during the standstill;
- both the minimum pressure peaks and the tendency during standstill seem confirm that the required equilibrium pressure at P7 is about equal to (and probably less than) 100kPa, so approaching the exact calculated value, without the assigned increase in favour of safety.

The above comments point out that the applied pressure could be higher than strictly required and, as consequence, there is an objective difficulty for the operator to attain the OAR (R=1 & stable pressure into the chamber).

Similar conclusions have been derived for the majority of the examined sections and the following comment can be generalised: there is an unusual "conflict" between the concurrent requirement of:

1) a safety margin, which is always compulsory in a geotechnical design and

2) the achievement of the "optimal" condition for the EPBS advancement.

Furthermore, if according to laboratory experiments $\sigma'_T \approx (\gamma' * H * k_O)$, a very low at-rest pressure coefficient (k_O) should be derived, even more than generally hypothesized for residual soils. A possible explanation could be also that in such complex geotechnical environment, the overall behaviour is locally governed by "rock-like" horizons, or, more probably, that (3D) LEM fits better the real equilibrium condition at tunnel face than stress analysis.

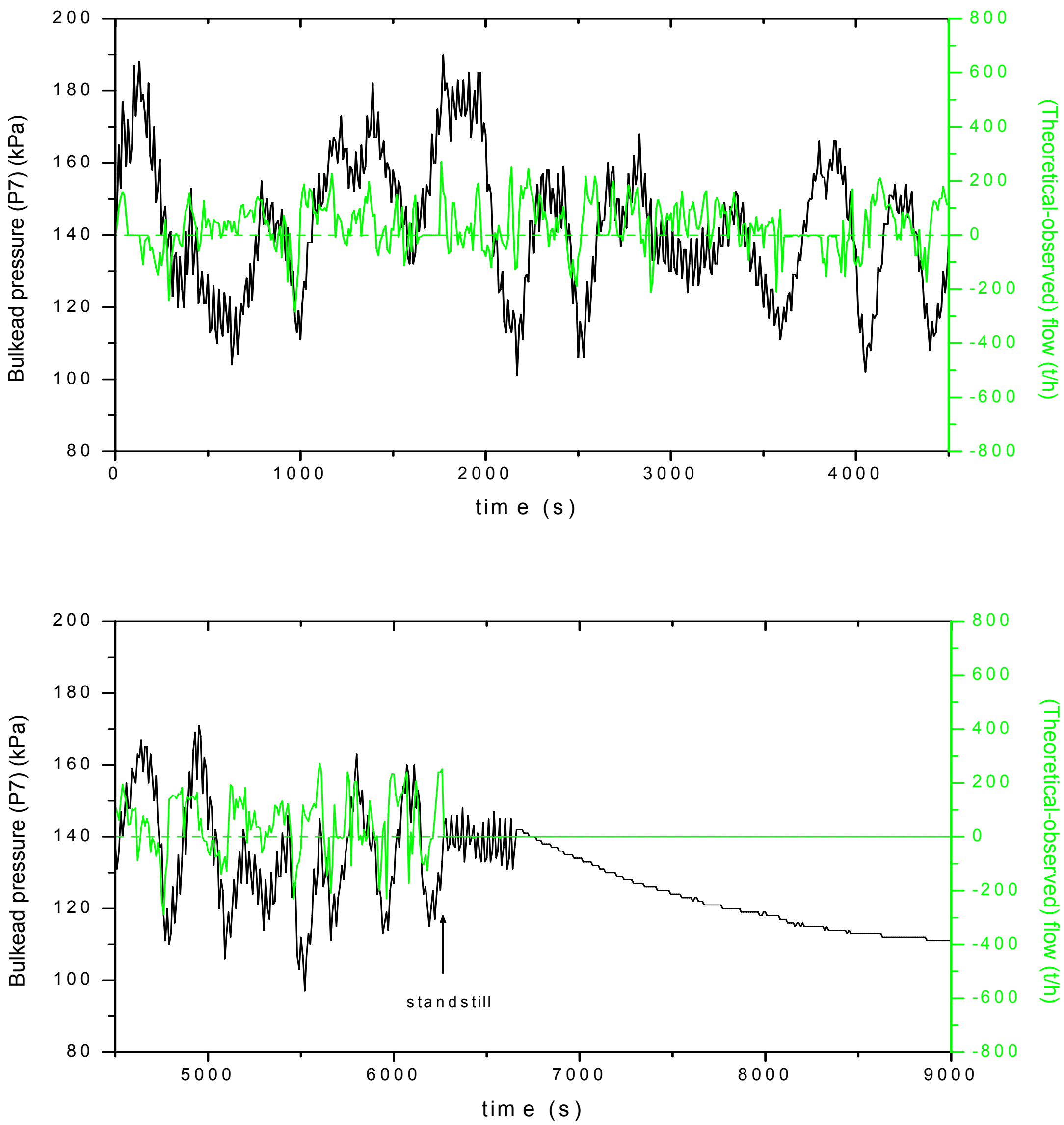


Fig.3.7: Bulkhead (P7) pressure and difference between the theoretical and the actual material flow through the screw conveyor.

4.CONCLUSIVE REMARKS

Some considerations about the correct definition of the face confinement pressure in mechanised tunnelling have been presented, with particular reference to Earth Pressure Balance (EPBS) shields.

Initially, some referenced LEM for evaluating the stability of the face have been presented and applied in practice for comparison. Then, the consequent problem of defining the adequate design face pressure has been dealt with, analysing both international practice and experimental research in laboratory (AFTES, 2001).

Some relevant concepts have been confirmed by these experiments: the optimal regime of advancement (OAR) for EPBS advancement involves both the balance of extracted vs. removed material and stable pressure condition in the working chamber; when these conditions are attained, the pressure applied by the EPBS is equalising the earth pressure at-rest. On the other hand, if the material actually extracted is less or more than the theoretical one, the passive or active state crops up in the ground, respectively.

As logical consequence, the adequate face pressure for achieving the OAR can be gradually assessed by means of rigorous control of these parameters, starting from a reasonably safe initial design value (σ_{Ti}). With specific reference to the applied confinement, it is important to point out that this control does not focus on the absolute values, but mainly on the stability of the pressure when R=1.

Especially for tunnel in urban environment, risk analysis can suggest to derive the effective σ'_{Ti} with different methods in function of the tunnel depth and, as pointed out by Barla (1994), of the related failure mechanism:

- for H<D: σ'_{Ti} should equalise the horizontal earth pressure at-rest, as derived from in situ stress analysis; given the related uncertainties, a crossed check of risk of blow-out failure is in any case recommended;
- for H>D: σ'_{Ti} can be reasonably determined by the described (3D) LEM, taking into account (particularly for L&D and A&K methods) an adequate safety margin, which incorporates geotechnical variabilities and uncertainties, as well as the possible pressure fluctuations in the working chamber.

The subsequent achievement of the OAR during the advancement of the EPBS can be complicated by the following practical restrictions:

1) the actual uncertainties on estimating the in-situ density and consequently the weight of the material to be excavated;
2) the actual accuracy of the weighting process on the EPBS itself, which is around 3-5% and can be hardly improved, and
3) the same usual criteria for geotechnical design and risk analysis, which impose the application of factors of safety.

The subject is evidently tricky and inciting, due to the concurrent requirement of an adequate margin of safety in designing and a optimal excavation process. Necessary, the possibility of a reasonable compromise is linked to the reduction of the described uncertainties by means of more and more reliable and precise technologies to check continuously the key control parameters (in situ-density of the material to be excavated, pressure in the working chamber and weight of the extracted material).

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