Shield tunnelling in sensitive areas: a new design procedure for optimisation of the construction-phase management

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ABSTRACT

The evaluation of the correct face-stabilizing pressure is a critical element in the various design and construction phases of a tunnel excavated by means of a TBM with EPBS (Earth Pressure Balanced Shield) in difficult geotechnical conditions. During the advancement of an EPBS, a combined use of analytical (probabilistic) and numerical methods is probably the most efficient approach to check continuously the actual conditions encountered and apply the counter-measures in a timely manner. An innovative, and well-applied, procedure for optimising the construction phase management is described in this article. The starting point of this procedure involves the verification of the results of numerical methods obtained from referenced analytical methods. In the first step of the procedure the results obtained through the (Caquot) analytical method are verified by means of a numerical method in order to evaluate the practical consequences in terms of development of deformations and plastic zone. In this manner, the assumed design risk is evaluated for the different methods and the solution that gives the best correspondence with numerical simulation is selected. Then, residual uncertainties and parametric variations are incorporated in the analysis and Monte Carlo simulation is used to calculate the statistical distribution of the face-stabilizing pressure and the design value is selected on the basis of an acceptable probability of failure. Finite Element Analysis FEM is finally performed on the basis of this value and results of back-analysis to verify the correspondence between numerical results and monitoring data. The application of the procedure to the twin tunnels underpassing the city of Bologna described here starts with determination of the face pressure (as a function of the stability of the face and the need for confinement of pre-settlements).

1. INTRODUCTION

Face stability is one of the fundamental factors in selecting the adequate control of excavation settings for an EPB. As a logical consequence, the evaluation of the stabilizing face pressure is a critical element for both the design and the construction phase. In spite of the importance of the subject, specific recommendations or technical norms are not available to provide guidance for the design. In practice, different approaches are often employed to evaluate the stability condition of the face and to assess the required stabilizing pressure. This present article deals with these aspects and gives a contribution to the base of theoretical-experimental and monitoring considerations, with particular reference to the realization by EPB of the high-speed railway hub in Bologna (Italy). The difficulties encountered during mechanized tunnelling in urban environment are summarized in Sec. 2. Some of the referenced analytical methods for evaluating the stability of face are presented and compared in Figure 2 of Sec. 3 for various friction angles and face pressures. The consequent problem of defining the adequate design face pressure is treated through numerical analysis, probabilistic methods and back-analysis data. Sec. 4 gives a practical example of this mixed approach, applied in a real case of tunnelling excavation with EPB shield.

2. EPBS TUNNELLING IN URBAN ENVIRONMENT: RISK AND PROBLEMS

Risk management is an essential part of the proper design and management of underground projects. The excavation design should aim to reduce the risk as low a level as is reasonably practicable. The risk management process includes various steps, that involve risk identification, risk assessment and risk response. The principal risks (or hazards) to the safety in urban tunnelling that are linked to the face pressure design can be identified as follows:

- unforeseen or unexpected ground conditions;
- variable and mixed face conditions (fine sand layer);
- ground loss/collapse at the face, causing inundation and/or large settlements;
- man-made obstructions or hazards to tunnelling, including utility services and unexploded bomb;
- collapse of the completed tunnel;
- human errors.

Risk Management is an essential part of the proper design and management of underground project. The determination of appropriate face pressure needs to be considered taking proper account of the uncertainties linked to the ground conditions, including groundwater and TBM design.

3. FACE PRESSURE EVALUATION

3.1 Analytical solutions

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. As reported by G. Russo (2003) in many cases the use of the limiting equilibrium methods (LEM) gives satisfactory solutions. LEM represents an important practical tool for design, especially when based on three-dimensional failure models. A comprehensive treatment of this subject is furnished by W. Broere (2001), Jancsez and Steiner (1994), Leca and Dormiuex (1990), Anognostou and Kovari (1996), Atkinson and Pott, COB (1996), Ribacchi (1994). Statically admissible solutions – based on lower and upper bounds theorems of plasticity – are normally considered to be more rigorous than the limit equilibrium solutions. Among statically admissible solutions we can mention the solutions by Caquot, (Caquot and Kerisel, 1956), these solutions are derived for 2D circular tunnel sections but can be easily extended to consider 3D spherical geometry. Caquot's model considers the equilibrium condition for material undergoing failure above the crown of a shallow circular (cylindrical or spherical) cavity. The material has a unit weight γ and a shear strength defined by Mohr-Coulomb parameters c and ϕ – the cohesion and the friction angle respectively. The distribution of vertical stresses before excavation is lithostatic and the ratio of horizontal to vertical stress is one. A support pressure p_s (e.g., provided by a liner) is applied inside the tunnel, while a surcharge q_s (infrastructures, embankments) acts on the ground surface. For the situation presented in Figure 1, Caquot's solution defines the value of internal pressure (p_s) as the minimum or critical pressure below which the tunnel will collapse. The Caquot generalised solution for dry conditions (which include the factor of safety *FS*), can be represented by the equation (1) developed by Carranza-Torres (2004):

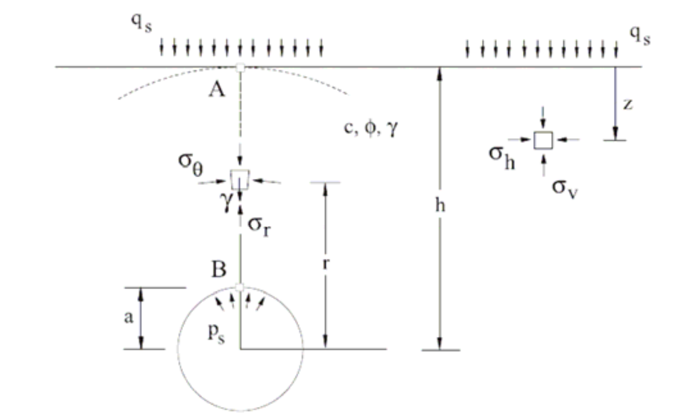

Figure 1. Caquot's model. (after Carranza-Torres, 2004).

$$\frac{p_s}{\gamma a} = \left(\frac{q_s}{\gamma a} + \frac{c}{\gamma a}\frac{1}{\tan\phi}\right)\left(\frac{h}{a}\right)^{-k\left(N_\phi^{FS}-1\right)} - \frac{1}{k(N_\phi^{FS}-1)-1}\left[\left(\frac{h}{a}\right)^{1-k\left(N_\phi^{FS}-1\right)} - 1\right] - \frac{c}{\gamma a}\frac{1}{\tan\phi} \qquad (1)$$

where: a is the tunnel radius; h is the axis depth below the surface; k is the parameter that dictates the type of excavation (1=cylindrical tunnel, 2= spherical cavity). It should be noted that equation (1) is valid only when the given Mohr Coulomb parameters lead to a state of limiting equilibrium – the situation in which the excavation is about to collapse. In general, the strength of the material will be larger than the strength associated with the critical equilibrium state of the cavity. The factor of safety FS can than be defined as " the ratio of actual Mohr-Coulomb parameters and critical Mohr-Coulomb parameters", as expressed in equation 2 (Strength Reduction Method, after Dawson, E. M., Roth, W. H. & Drescher, A. 1999).

$$FS = \frac{c}{c^{cr}} = \frac{\tan\phi}{\tan\phi^{cr}} \qquad (2) \qquad\qquad N_\phi^{FS} = \frac{1+\sin\left(\tan^{-1}\dfrac{\tan\phi}{FS}\right)}{1-\sin\left(\tan^{-1}\dfrac{\tan\phi}{FS}\right)} \qquad (3)$$

Equation (1) is valid for any given combination of parameters, whether or not the excavation is in the state of limiting equilibrium, and allows computation of a factor of safety for the tunnel. In Figure (2) different analytical methods above mentioned are confronted, in particular referring to the Caquot's solution with FS=2. The progressive reduction of the face pressure with the growing of the friction angle (ϕ) value can be observed. Moreover, on the same diagram, potential stability conditions of the face are evidenced: they pass from an elastic deformation condition to the complete collapse, for progressive reduction of p_S. From the static point of view, better conditions result applying a face pressure equal to the earth pressure at rest (K_0), assumed as upper limit of application for the stability pressure design.

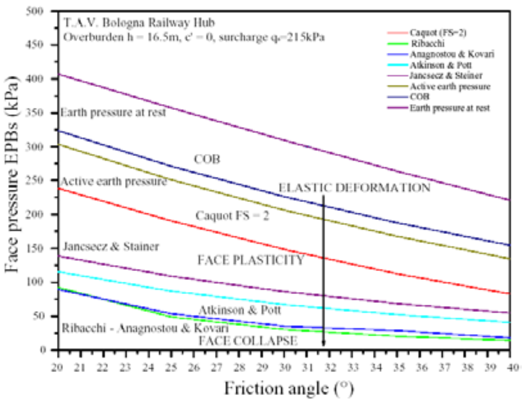

Figure 2. Face pressure vs. friction angle.

3.2 Numerical methods: 2D and 3D analysis

If on one side analytical methods concur a simple and fast appraisal of the face pressure, useful to limit/avoid the complete collapse of the tunnel, on the other hand they do not say anything about the development of the deformation gradient around the excavation and its propagation towards the surface. The permissible movements entity is nearly always – especially in urban areas – linked to the presence of pre-existent infrastructure that must be persevered in their stability and functionalities during the entire tunnel constructive process. The admissible settlement is a critical point in the design process, analytical solution do not take into account all the variables that are

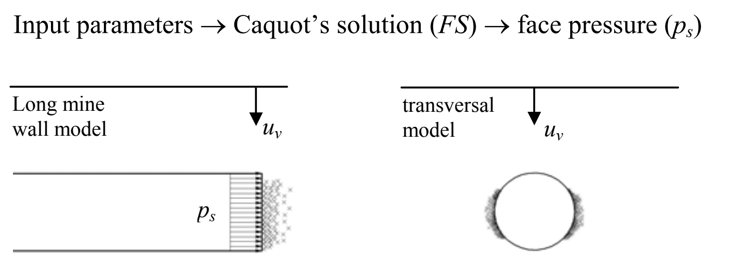

Figure 3. Face pressure. Mixed analytical/numerical method

encountered during the excavation path, exceptional boundary conditions like different geology or suspended water bearing are not simulated, for this reason it be necessary to use 2D or 3D numerical models as shown in Figure 3.

3.3 Comparison of analytical and numerical solutions.

During the EPB advancement, the use of a combination of analytical/numerical methods is probably a better approach for estimating the stress/strain conditions. It allows to make a timely application of the appropriate countermeasure to guarantee the stability of the face and to limit the propagation of settlements towards the surface. A FEM numerical model approach is outlined in Figure 3. The model explains the functional relationship between the face pressure and the value of mobilized strains (or maximum shear strain) around the excavation profile and along the ground surface. This is fundamental to detect the increase in the probably failure through the shield and the surface. Shear bands could connect the excavation face with ground surface, causing damage and collapse of superficial structures. The following section discusses the problem of the adequate evaluation of the face pressure FS will be treated in the case of a preliminary approach to the excavation.

3.4 Probabilistic "Monte Carlo" analysis for determining the probability of failure

Using analytical solutions it is possible to include in the evaluation of face pressure the variability of the input parameters (e.g. geotechnical and geometrical conditions) through the Monte Carlo probabilistic method; this is not allowable with numerical methods. In this kind of analysis, once the factors of safety have been calculated with the Caquot's solution, for every face pressure value (p_s) it is assigned the respective probability that the chosen factor of safety is not reached or overcome, as shown in the graph in figure 4, where the reference value of *FS* is 2. In particular for every value of face pressure (p_s) it has been executed a probabilistic analysis with variation of the representative parameters of input with the aim to estimate the probability of *FS*<2; as an example for pressure of approximately 1.10bar, condition *FS*<2 has one probability of 40% to take place. Such diagram is referred to one particular analysis condition, in terms of overburden (h), tunnel radius (a), soil volume weight (γ), assumed as deterministic values; geotechnical parameters (e.g.: ϕ) have been made to vary by means of laws of triangular distribution in a determined representative range of variation. Although he is fundamental to operate numerical analyses in order to estimate the extension of the plastic zone and therefore of the influence of the disturbance induced from the excavation in surface, the proposed analysis allows to formulate one first hypothesis on the values of the ranges of face pressure (p_s) with which to operate, concurring of having sensibility of the relative degree of risk-security. Moreover they can supply a departure point in order to estimate the pressure to adopt for the numerical simulations, and eventually if these last ones could give admissible settlement, to become design pressures.

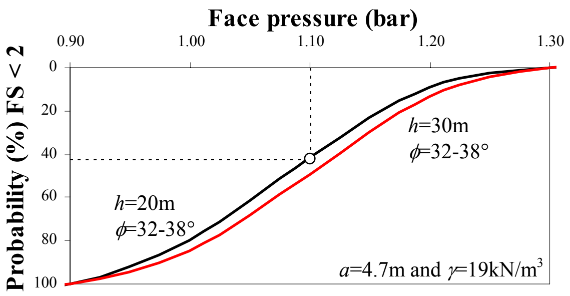

Figure 4. Face pressure and probability (*FS*)

4. PRACTICAL APPLICATION: TWO METROPOLITANS PARALLELS BORED

4.1 General description and Geological setting

The realisation of the High Speed Rail System Project in Italy includes intense urbanised areas. In 2000, the joint venture S.Ruffillo was awarded one of the most critical lots, under passing the city of Bologna, starting from the S.Ruffillo quarter, south of the city, up to the new Rail Central Station of

Bologna, downtown. The 7km long, single truck, twin EPB tunnels called “Pari” and “Dispari”, started in July 2003 and November 2003 respectively, introducing a longitudinal distance between the front faces of the two machines of about 500m. The two tunnels have a diameter of excavation of 9.4m, with an overburden (*h*) between 15 – 21m, without considering the overlying 10m-railroad embankment. For most of the alignment, the interaxial distance between the two tunnels is 15m, this means that the distance between the outer linings is around 5.9m. This distance has been chosen in order to minimise the settlement along the width and at to leave an adequate pillar between the tunnels to avoid any alignment problems. The tunnels pass sandy and clayey sediments – Pliocenic Clay and Clayey Sands – below the water level, from S.Ruffillo entrance for 2100m, then up to Stazione Camerone they go through manly gravely and sandy alluvial sediments, either below and above water level, belonging to Savena River deposits.

4.2 Subsidence monitoring and consequential evaluation of the design stabilizing face pressure.

Figure 5 illustrates the relationship for various sections between medium face pressure and the pre-settlement volume loss. It is interesting to note the face pressure influence on the recorded volume loss (V_s) at the front. It is important to observe that the volume loss occurring in the monitored Bologna granular soil, with an excavation face pressure higher than 1.4bar, is limited to 0.1%. It is important to observe that in this case, the correlation between applied face pressure and recorded volume loss is not dependent from the eventual physiological component of settlements, due to the different diameter between the cutter-head and the shield. From an analysis of the recorded values of the displacements in several sections, it has been possible to estimate the dimension of settlement as a percentage of the measured maximum displacement, in this case:

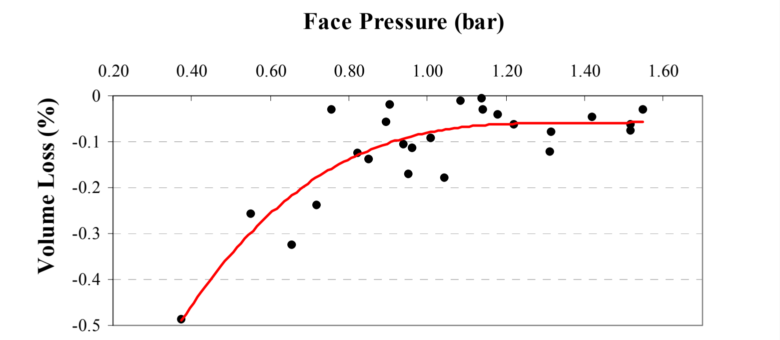

Figure 5. Volume Loss vs. Face pressure

- the 20-30% when the shield face is on the measured section;
- the 30-40% when the shield face is 5-10 meters after the measured section;
- 60% when the shield face is 20 meters after the measured section.

It is specified that analyses do not take into account the temporal delay due to the reologic behaviour of sands, and to the propagation of the strain perturbation from the shield to the measuring instruments on surface. To identify the correct Safety Factor to attribute to Caquot’s analyses, it was initially estimated as *FS*=2 on the basis of the monitored face pressure (TBM “pari”) when the volume loss was less than 2. The obtained *FS* values were correlated with the recorded volumes loss in the same sections as reported in Figure 5. The variability ranges of the safety factor that supplied values of volume loss less than 0.1% have been estimated and illustrated in Figure 6.

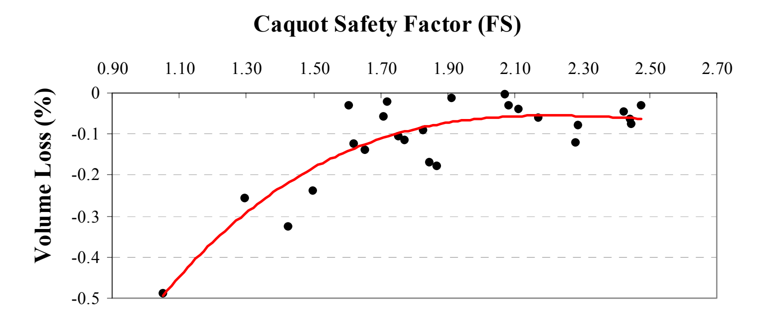

Figure 6. Volume Loss vs. Factor of Safety (Caquot’s solution)

4.3 Back-analysis: Comparison of results of numerical analysis and monitoring

After the range of the front pressure variation (p_s) has been identified, it is necessary to evaluate its reliability through numerical analyses as reported in Sec. 3. Using numerical analyses, it is possible to estimate the magnitude of surface settlements, the shear stress evolution and the potential propagation of shear band (shift) toward the surface. A numerical analysis result, performed by using the computational software Phase2® (Rocscience), is shown in Fig. 7 where the failure zone and the volume loss can be observed. The analysis has been conducted using the face pressure values that guaranteed a minimum factor of safety of 2, in accordance with the Caquot's model. The recorded surface settlements (i.e. 0.5cm as pre-settlement values) will have to be increased on the basis of back-analysis percentages (Sec. 4.2), in order to define the final/total displacement. This value will be directly confronted with the allowable settlement for the pre-existing structures, in order to conserve their functionality and integrity. In this way numerical analysis becomes a further instrument to validate the choice of the appropriate face pressure value.

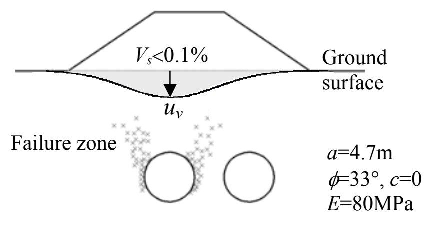

Figure 7. Numerical analysis Phase2

5. CONCLUSIONS

A mixed statistical-numerical approach can supply, in the first excavation phase of a tunnel with EPB shield, a correct instrument for estimating the initially stabilization pressure to be applied at the face, guaranteeing the pre-defined or acceptable safety margins (*FS*>2). Such pressure will be reviewed and eventually corrected in coherence with the back analysis results. The planning of the support pressure will have to be designed for stretches of the tunnel with homogenous behaviour from the geological-geotechnical point of view, so as to assign a constant pressure value to maintain until to the successive paths encountered, where a new phase of planning will be started.

REFERENCES

Anagnostou, G. & Kovari, K., 1993. "Face stability conditions with Earth-Pressure-balanced Shields." Tunnelling and Underground Space technology, Vol. 11, No. 2, pp. 165-173.

Caquot, A. & Kerisel, J., 1956. "Traité de Mécanique des Sols." Paris ; Gauthier-Villars.

Carranza-Torres, C., 2004. "Computation of Factor of Safety for Shallow Tunnels using Caquot's Lower Bound Solution." In publication.

Dawson, E. M., Roth, W. H. & Drescher, A. 1999. "Slope stability analysis by strength reduction." Geotechnique 49(6), 835–840

Gaj, F., Guglielmetti, V., Grasso, P. & Giacomin, G. 2003. "Experience on Porto – EPB follow-up." Tunnels and Tunnelling International, December 2003, 15-18.

Leca,E. & Dormieux, L., 1990. "Upper and lower Bound solutions for the face stability of shallow circular tunnel in frictional material". Géotechnique." Vol. 40, n. 4 pp. 581-606.

Jancsecz, S. & Steiner, W., 1994. "Face support for a large Mix-Shield in heterogeneous ground conditions". Tunnelling 94, London.

Minguez, F., Gregari, A. &. Guglielmetti, V. 2005. "Demonstrating "best practice" in EPB management." Tunnel and Tunnelling International - Nov. 2005

Romano, M.C., 2005. "EPBS twin tunnels excavation: case study the High Speed Rail system in Bologna." Master Thesis in Tecniche per la progettazione e la valutazione ambientale.

Russo, G., 2003. "Evaluating the required face-support pressure in EPBs advance mode." Gallerie e Grandi Opere Sotterranee n.71 - Dicembre 2003.

Terzaghi, K., 1967. "Theoretical Soil Mechanics." New York: John Wiley & Sons, Inc.