Assessing Face Stability Conditions for Shallow Tunnels in Soft Ground through the Tunnel Stability Factor

Spyridon Konstantis[1]

[1] ARUP Geotechnics and Tunnelling, UK

ABSTRACT

The contribution discusses the use of the Tunnel Stability Factor (TSF) for the preliminary assessment of face stability for shallow tunnels in homogenous soft ground. The required face support pressure to ensure stability conditions is estimated probabilistically using a limit equilibrium model (Anagnostou & Kovari method), an approach based on the convergence-confinement method and a combination of the two. The random variables in the parametric studies are the tunnel geometrical and the ground geotechnical characteristics. The ground shear strength parameters c and φ are assumed to be uncorrelated variables and are considered design values in accordance with EC7. The groundwater conditions Hw are treated both as a random and an independent variable and no destabilising seepage forces are considered on the tunnel face. The probabilistic analyses were carried out with the software @Risk™ and Monte Carlo simulation was used as the sampling technique for the random variables. The definition of the TSF is based on the correlation coefficients of the input random variables for the output required face support pressure P. In the present work, dimensionless graphs were produced and their analytical expressions were derived that correlate the required ratio P/c to ensure face stability conditions with the TSF. Applications for typical shallow tunnels in soft ground are presented and the results are discussed.

1 INTRODUCTION

Face stability is a key factor for shallow tunnels excavated through challenging and difficult ground conditions and in many cases under severe environmental constraints. Tunnel face instability may lead to unacceptable relaxation of the advance core (Lunardi, 2000) and in extreme cases to partial or total face failure (ITA-AITES, 2006). The factors that influence the face stability conditions are associated with the geometrical dimensions of the tunnel and the hydro-geological/geotechnical characteristics of the ground.
The assessment of face stability conditions and necessary support pressure requires in principal 3D numerical analyses. However, these analyses have a number of certain drawbacks mainly associated with complicated input preparation and output presentation, increased computational effort, multiple simulation stages and incompatibility between improved accuracy and level of knowledge of ground conditions (Kavvadas, 2005).
In the literature, there are many available analytical methods that can be used to assess the face stability conditions. A summary can be found in (Guglielmetti et. al., 2007). Most of these methods are based on limit equilibrium solutions or the upper and lower bound theorems of plasticity.
In conceptual studies and preliminary design stages the available geotechnical data are in general limited, leading to an increased uncertainty associated with the design input parameters and in many cases parametric analyses have to be carried out. In such cases the use of simple and practical probabilistic assessment tools, based on the available methods in

the literature, would lead to a significant reduction in numerical effort/analytical studies and associated time and cost.

2 FACE STABILITY ASSESSMENT METHODS

In the present contribution three different analytical methods have been used: a) the Anagnostou & Kovari method (Anagnostou & Kovari, 1994 & 1996) b) an approach based on the convergence-confinement method (Kavvadas, 2005) and c) a combination of the two (Kavvadas, 2002).

2.1 Anagnostou & Kovari (A&K) method

This limit equilibrium method was originally introduced by Horn (1961) and has its basis on the silo theory by Janssen (1895). The failure mechanism comprises a 3D wedge on the face and a prism above it. The loads acting on the wedge are its self weight, the resulting normal and shear forces along the failure (slip) surfaces, the applied face support pressure and the overburden silo load imposed by the overlying prism. The methodology for defining the necessary face support pressure is based on the 3D wedge limit equilibrium and the process is iterative. For a detailed description of the method, refer to (Anagnostou & Kovari, 1994 & 1996).
The method can be used to assess the face stability conditions and the required face support pressure for both mechanised (TBM) and conventional (SCL) tunnelling. The original method of A&K assumes that the ground on the face wedge is homogenous. Anagnostou & Serafeimidis (2007) and Serafeimidis et. al. (2007) have extended the method to account for heterogeneous tunnel faces.

2.2 Convergence – Confinement method

The excavation of a tunnel induces ground deformations and causes changes in the stress regime of the surrounding ground as a result of the gradual reduction of the in situ stress Po. This stress relaxation can be expressed as:

$$Pi = (1-\lambda)Po \tag{1}$$

In the above equation, Pi represents the self supporting mechanisms of the ground with the form of a fictitious internal support pressure, Po is the in situ geostatic stress at tunnel level and λ is the deconfinement ratio that can be defined through a convergence-confinement analysis (Kavvadas, 2005). Ahead of the advance core where the stress state is geostatic λ equals to zero and increases gradually to unity at about one and one half tunnel diameters behind the face for the case of a fully unsupported tunnel (Hoek, 2007).
In the advance core of a tunnel with unsupported face, the geostatic lateral confining stress σ3 in the direction of the tunnel advance is gradually reduced and becomes zero at the face. In a simplified approach, the stress state at the unsupported face area represents uniaxial compression conditions (Kavvadas 2005).
In this case, the initial safety factor FSo of the tunnel face can be conservatively expressed as:

$$FSo = \frac{\sigma_{cm}}{\sigma_1} = \frac{2}{(1-\lambda)Ns} \tag{2}$$

where $Ns=2Po/\sigma_{cm}$ is the overload factor, $\sigma_{cm}=2c\cos\varphi/(1-\sin\varphi)$ is the ground strength, c is the ground cohesion, φ is the ground friction angle and $\sigma_1=(1-\lambda)Po$ is the relaxed overburden

stress at distance x=Dtan(45-φ/2)/2 ahead of the face, assuming a circular tunnel with diameter D and a failure mechanism according to the A&K method.
If face support pressure σ_3 is applied the ground strength increases to $\sigma_c=\sigma_3\tan^2(45+\varphi/2)+\sigma_{cm}$, the stress state at the face area represents triaxial conditions and the improved safety factor FS becomes (Kavvadas, 2005):

$$FS=\frac{\sigma_c}{\sigma_1}=FSo+\frac{1}{(1-\lambda)}\left(\frac{\sigma 3}{Po}\right)\tan^2\left(45+\frac{\phi}{2}\right) \tag{3}$$

The above equation (3) assumes that λ is not affected by the application of the face support pressure σ3. In a conventionally excavated tunnel where the face is, for instance, reinforced with fibreglass nails, this assumption is acceptable as in this case the crown settlement of the tunnel is only slightly reduced (Kavvadas, 2005). In a mechanically (TBM) excavated tunnel, however, λ depends on the confinement level applied by the TBM (Aristaghes & Autuori, 2003). The critical value of the deconfinement ratio λcr is defined as (Kavvadas, 2002):

$$\lambda_{cr}=1-\left(\frac{2}{1+k}\right)\left(\frac{Ns-1}{Ns}\right) \tag{4}$$

where $k=\tan^2(45+\varphi/2)\geq 1$ and the following cases are considered:

- If λcr≥1 (elastic ground) or if {λcr <1 (elastoplastic ground) and λ≤λcr}, then no plastic zone is developed around the tunnel (Elastic conditions)
- If λcr <1 and λ>λcr (elastoplastic ground), then plastic zone is developed around the tunnel (Plastic conditions)

This simplified approach is, in principal, conservative as it does not account for the shear resistance at the lateral failure surfaces and thus the horizontal stresses σ_2. Moreover, the C-C method is based on a plain strain analysis of a circular tunnel under isotropic stress conditions. If ground water exists, the water flow through the tunnel face induces destabilising seepage forces with intensity $f = i\gamma_w$, where i is the hydraulic gradient and γ_w is the ground water unit weight, resulting in the reduction of σ3 (Kavvadas, 2002). The seepage forces have also a significant effect on the deconfinement ratio λ as they modify both the Ground Reaction Curve and the Longitudinal Deformation Profile (Lee & Nam, 2006).

2.3 Combination of A&K and Convergence-Confinement method

In this method (Kavvadas, 2002), which is based on the A&K method, the overburden load acting on the wedge is the relaxed overburden stress $\sigma_1=(1-\lambda)Po$ at distance x=Dtan(45-φ/2)/2 ahead of the face and the shear stress acting along the lateral failure surfaces is:

$$\tau_f=c/F+K\sigma_v\tan\phi/F\,S \tag{5}$$

where K is the earth pressure coefficient, σ_v is the vertical stress at tunnel level and FS is the safety factor.
The nature of K depends primarily on the stress relaxation in the advance core. In front of the advance core, the stress state represents conditions at rest Ko. Due to the radial pre-convergences that start to develop in the advance core, the stress state could be assumed to represent active earth pressure conditions Ka. In the present work, however, Ko was adopted in the parametric analyses. The original method of A&K considers 'Ko'=0,4 for the wedge at tunnel level. Under ground water conditions, an effective stress analysis (drained conditions) has to be carried out and the seepage forces have to be taken into account, as described in Anagnostou & Kovari (1996).

3 TUNNEL STABILITY FACTOR

The Tunnel Stability Factor (TSF) has been introduced by Mihalis et. al. (2001) for the assessment of the behaviour of underground openings in weak rock conditions and is defined as:

$$TSF = \frac{\sigma_{cm}}{\gamma H^{a} D^{1-a}} \tag{6}$$

where σ_{cm} is the ground strength, γ is the ground unit weight, H is the height of overburden and D is the equivalent tunnel diameter. The dimensionless parameter $\alpha \leq 1$ defines the relative contribution of H and D to the considered tunnel behaviour.

4 PROBABILISTIC ANALYSES

The probabilistic analyses were performed with the software @Risk™ (www.palisade.com) and Monte Carlo simulation was used as the sampling technique for the uniformly distributed random variables.

The following ranges of the random variables were examined:

- Tunnel diameter D=4m-10m
- Overburden height (ground level to tunnel crown) H=5m-30m
- Cohesion c=5KPa-25KPa
- Friction angle $\varphi=20^{o}-35^{o}$
- Earth pressure coefficient at rest Ko=1-sinφ
- Dilation angle $\psi=0^{o}-\varphi^{o}/6$
- Elasticity modulus E=15MPa-60MPa
- Poisson's ratio v= 0,15-0,25
- Ground unit weight γ=18-22 KN/m^3

According to Mollon et. al. (2009), the assumption of negative correlation between the shear strength parameters c and φ gives greater reliability of the tunnel face stability. In the present work, however, c and φ were assumed to be uncorrelated.

The ground water conditions Hw were taken into account only in the A&K method and were treated both as random and independent variable (Mihalis et. al., 2009). In all the examined cases, no destabilising seepage forces were considered on the tunnel face.

5 SAFETY FACTOR CONSIDERATIONS

The analyses were performed for unity safety factor (FS=1). This approach is in accordance with the general framework of Eurocode 7 (EC7) that requires Design Resistance Rd≥ Design Effect of Actions Ed.

In essence, the tunnel face can be considered as a vertical slope. In EC7 the design requirements for the overall slope stability are defined through the Geotechnical (GEO) and Structural (STR) Limit States that concern failure (Ultimate Limit State) or excessive deformation (Serviceability Limit State) of the ground and/or of any supporting structure, respectively. The methods adopted here for the assessment of face stability conditions do not consider the structural contribution of the support systems but only the resulting pressure, hence the STR limit state is not considered. Moreover, the methods are based on failure mechanisms rather than deformation driven modes. Consequently, the overall stability of the tunnel face can be considered here as a GEO ULS situation. The ground shear strength parameters c and φ are considered design values in accordance with EC7.

6 RESULTS

In the following results, the dimensionless parameter α≤1 of the TSF was derived through an iterative process for the maximisation of the coefficient of determination R^2 of the scatter plots. In this process, the correlation coefficients (relative contribution) of the input random variables H and D for the output support pressure P that has to be applied on the tunnel face to ensure stability conditions were used. The support pressure P refers to the tunnel centreline.

6.1 Anagnostou & Kovari (A&K) method

6.1.1 Dry conditions

The results of the probabilistic analyses are presented in Figure 1. It can be observed that the trend is very well defined and no face support pressure P is required to ensure stability conditions for TSF (α=0.07) > 0.35.

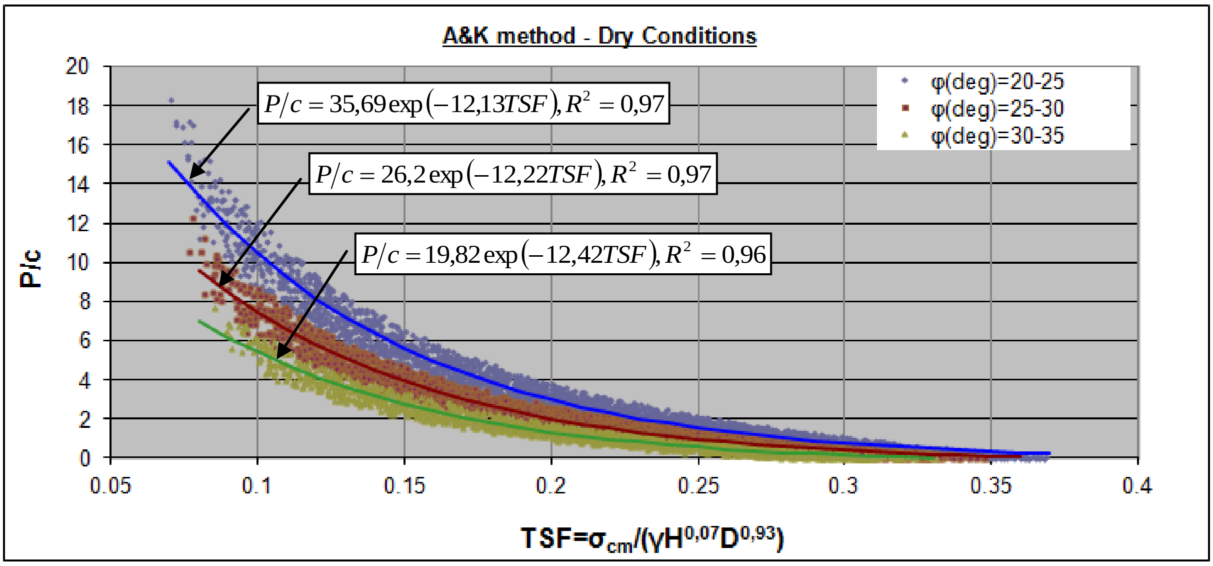

Figure 1 A&K method – Dry Conditions

6.1.2 Ground Water Conditions

In order to define the TSF for the case of ground water conditions, probabilistic analyses were carried out for ground water heights Hw={H, 3H/4, H/2, H/4, H/8). The ground water height Hw is measured from the ground water table elevation to the tunnel crown, i.e. Hw=H means that the ground water table is on the ground surface. In the case of Hw≠0, the dimensionless parameter α of the TSF can be expressed as a function of Hw/H (Konstantis, 2011):

$$a = \ln\frac{(Hw/H)}{0.0356}\Big/3.8833, Hw \neq 0, R^2 = 0.9974 \qquad (7)$$

In the results presented in Figure 2, Hw was treated as a uniformly distributed random variable. Even though the average trend is well defined, it can be observed that the range of P/c is relatively wide for TSF values smaller than 0.45 and the scatter increases with the reduction of the TSF. The smaller the TSF, the wider the range of required P/c, thus the bigger the uncertainty and the lower the reliability of this probabilistic approach.

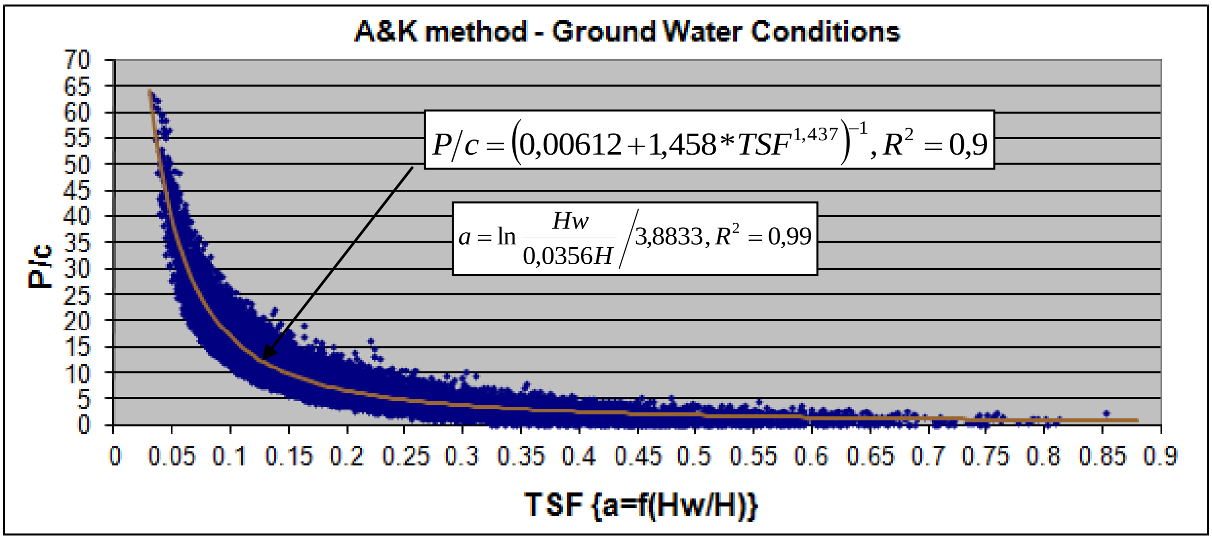

Figure 2 A&K method – Ground Water Conditions

In order to reduce the range of P/c for low values of TSF, Hw was also treated as an independent variable (Mihalis et. al., 2009) and the results are presented in the following Figure 3 for different ratios of Hw/H. The range of P/c can be further reduced by considering ranges of φ for ratios of Hw/H (Konstantis, 2011).

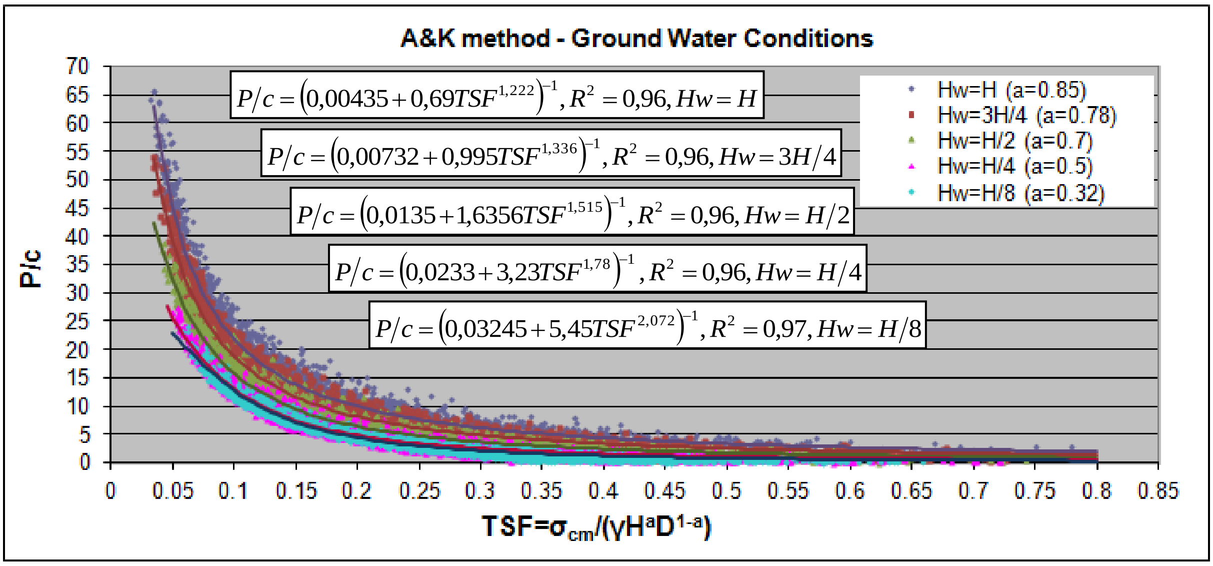

Figure 3 A&K method – Ground Water Conditions for ratios of Hw/H

6.2 Convergence – Confinement (C-C) method

6.2.1 Elastic conditions

The results are presented in the following Figure 4 where it can be observed that no face support pressure P is essentially required for TSF (α=0,98) greater than 0.6.

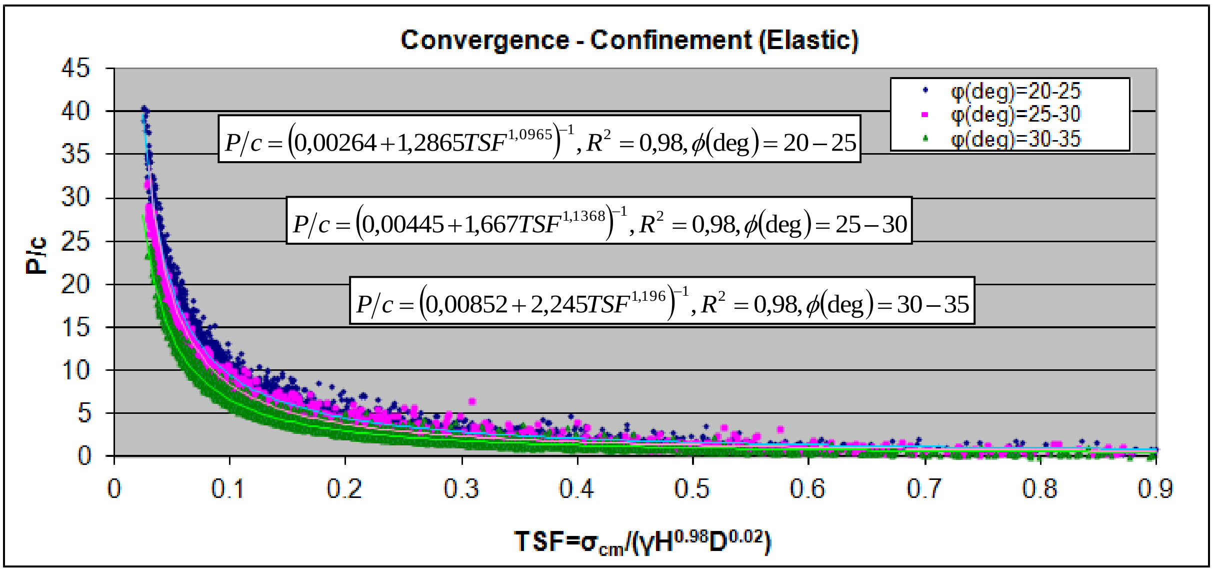

Figure 4 Convergence – Confinement method under Elastic Conditions

6.2.2 Plastic conditions

For the plastic conditions it was not possible to correlate the ratio P/c with the TSF. However, λ can be correlated with the overload factor $Ns=2Po/\sigma_{cm}$ and the necessary support pressure P can then be estimated as a percentage of the in situ stress Po (Konstantis, 2011). As λ increases, P decreases. However, high values of λ correspond to high values of Ns which in turn are associated with tunnel cross sectional instability. Consequently, the convergence - confinement method under plastic conditions should be used with extreme care for the assessment of face stability conditions and must be accompanied with cross sectional analysis to evaluate the overall behaviour and stability conditions of the tunnel.

6.3 Combination of A&K and C-C method under Elastic conditions

The results are presented in the following Figure 5.

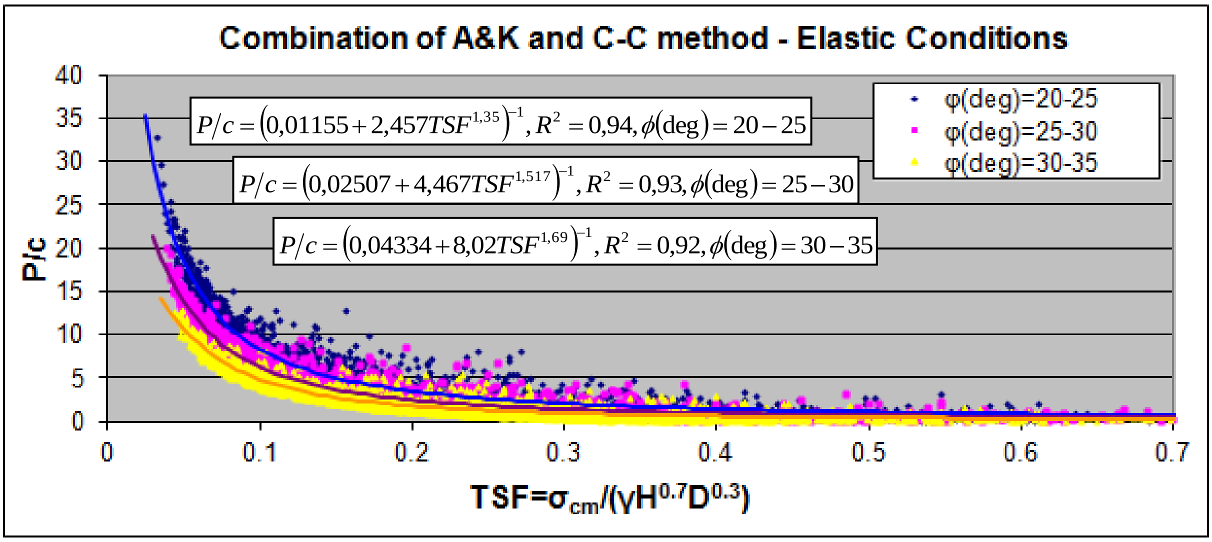

Figure 5 Combination of A&K and C-C method under Elastic conditions

7 APPLICATIONS

The applications concern two cases (the values in *italics* denote the *case 1* and the values in **bold** the **case 2**) of a typical tunnel of D=*8,7m* (**7.5m**) diameter with an overburden height above the tunnel crown of H=*16m* (**20m**), excavated through a ground mass with c'=*12KPa* (**15KPa**) and φ'=*26°* (**33°**). The ground unit weights γ=20KN/m^3 and γ'=11KN/m^3 as well as the elasticity modulus E and Poisson's ratio v are the same for the two cases. For the A&K method both dry and ground water conditions are investigated, whereas for the C-C method and the combination of A&K and C-C method only dry conditions are considered.

7.1 Dry Conditions

According to Anagnostou & Kovari (1996), the required face support pressure to ensure stability in dry conditions is equal to $s = Fo\gamma dD - F1c$, where Fo and F1 are dimensionless coefficients that depend on the friction angle φ and the ratio H/D, c is the ground cohesion and γd is the dry unit weight of the ground.
For *case 1*, Fo=0.32 and F1=2.65 and hence s=23.88KPa according to A&K method (deterministic value). From Figure 1, the required face support pressure is P=23.7KPa. Following the convergence-confinement method under elastic conditions from Figure 4, the required face support pressure is P=76.8KPa, whereas the combination of A&K and C-C method from Figure 5 gives P=45.9KPa.
For **case 2**, Fo=0.18 and F1=2 leading to s<0 according to A&K method (deterministic value), hence the face is stable without the application of support pressure. From Figure 1, the required face support pressure is P=4.15KPa. Following the convergence-confinement method under elastic conditions from Figure 4, the required face support pressure is P=67KPa, whereas the combination of A&K and C-C method from Figure 5 gives P=29.5KPa.
As expected, the C-C method under elastic conditions gives higher values compared to the combination of A&K and C-C method. The probabilistic results P based on Figure 1 are, in general, in very good agreement with the deterministic values s from the A&K method.

7.2 Ground Water Conditions

In the case of ground water conditions, the ground water height from the ground water table elevation to the tunnel crown is assumed to be Hw=*12m* (**10m**). The ground water pressure Pw is measured on the tunnel centre line and no destabilising seepage forces are taken into consideration on the tunnel face.
According to Anagnostou & Kovari (1996), the required effective face support pressure to ensure stability for the case of no head difference (Δh=0) is equal to $s' = Fo\gamma' D - F1c$, where Fo and F1 are the same dimensionless coefficients as above that depend on the friction angle φ, the ratio H/D and the water height, c is the ground cohesion and γ' is the buoyant unit weight of the ground.
For *case 1*, Fo=0.3 and F1=2.65 and s'<0. The water pressure Pw on the tunnel centre line is Pw=γw(Hw+D/2)=163.5KPa. Therefore, s=s'+Pw=163.5KPa. From Figure 2, where Hw is treated as a random variable, we get α=0.78 for Hw=3H/4 and P=133KPa. From Figure 3, where Hw is treated as an independent variable, we get P=155KPa.
For **case 2**, Fo=0.18 and F1=2, leading to s'<0. The water pressure Pw on the tunnel centre line is Pw=γw(Hw+D/2)=137.5KPa. Therefore s=s'+Pw=137.5KPa. From Figure 2 we get α=0.7 for Hw=H/2 and P=111KPa. From Figure 3 we get P=107KPa.
In the case of ground water conditions, the probabilistic results (P) are in relatively good agreement with the deterministic values (s) and in any case they provide a correct order of magnitude of the required support pressure. Apparently, the difference between P and s will decrease for reduced ranges of input random variables. For a conventionally (SCL) excavated tunnel, the face support measures -apart from ground freezing- do not provide active counteraction of the ground water pressures. Therefore, this probabilistic method for

the case of ground water conditions applies to mechanically (TBM) excavated tunnels with closed face shields.

8 CONCLUSIONS

The present contribution demonstrates the important role of the Tunnel Stability Factor (TSF) on the probabilistic, preliminary assessment of face stability conditions for shallow tunnels in various homogenous soft ground conditions. The probabilistic analyses were carried out with the use of Monte Carlo simulation techniques and the assessment was based on three different analytical methods.
Despite the different base and assumptions of the analytical methods, the results present a very well determined trend and validate the importance and usefulness of the TSF. Dimensionless graphs with analytical expressions have been produced that can be used in conceptual studies and preliminary design stages, saving a significant amount of effort and resources.
The methodology described could be applied to the various analytical methods available in the literature to produce similar graphs and analytical expressions, with the use of the TSF, for the preliminary assessment of face stability conditions and required support pressure.

REFERENCES

Anagnostou, G. & Kovari, K. 1996. Face stability conditions with Earth Pressure Balanced shields. Tunnelling and Underground Space Technology, Vol. 11, No. 2, pp.165-173

Anagnostou, G. & Kovari, K. 1994. The face stability of slurry-shield driven tunnels. Tunnelling and Underground Space Technology, Vol. 9, No. 2, pp.165-174

Anagnostou, G. & Serafeimidis, K. 2007. The dimensioning of tunnel face reinforcement. In: Bartak, Hrdina, Romancov & Zlamal (eds) ‘‘Underground Space – the 4th Dimension of Metropolises’’, 291-296. Taylor & Francis Group, London

Aristaghes, P. & Autuori, P. 2003. Confinement efficiency concept in soft ground bored tunnels. (Re)Claiming the underground Space, Saveur (ed.). Swets and Zeitlinger, Lisse, ISBN 90 5809 542 8

Eurocode 7, Geotechnical design - Part 1: General rules (BS EN 1997_1.2004)

Guglielmetti, V., Mahtab, A. & Xu, S., Geodata S.p.A., Turin, Italy. 2007. Mechanised Tunnelling in Urban Areas – Design methodology and construction control, Taylor & Francis

Hoek, E. 2007. Practical Rock Engineering

Horn M. (1961), Horizontaler Erddruck auf senkrechte Abschlussflaechen von Tunneln, In : Landeskonferenz der ungarischen Tiefbauindustrie , Budapest (German translation, STUVA, Duesseldorf)

ITA-AITES Report 2006 on Settlements Induced by Tunnelling in Soft Ground. Tunnelling and Underground Space Technology 22 (2007) 119-149

Janssen H. A. (1895). Versuche uber Getreidedruck in Silozellen, Zeitschrift des Vereins deutscher Ingeniuere, Band XXXIX, No. 35, 1045-1049 (in German)

Kavvadas, M. 2005. Numerical Analysis in the design of Urban Tunnels. Keynote Lecture. The 11th International Conference of IACMAG, Torino, 19-24 June, 2005

Kavvadas, M. 2002. Design of Underground Structures. University Notes. Specialised Master Design and Construction of Underground Works. National Technical University of Athens

Konstantis, S. 2011. Assessing Face Stability. Tunnels and Tunnelling International, September 2011

Lee, In-Mo. & Nam, Seok-Woo. 2006. Seepage Force Considerations in Tunnelling. International Symposium on Underground Excavation and Tunnelling. 2-4 February 2006, Bangkok, Thailand

Lunardi, P. 2000. Design and constructing tunnels-A.DE.CO.-RS approach. Supplemento a Tunnels & Tunnelling International. Maggio. 2000

Mihalis, I., Kavvadas, M. & Anagnostopoulos A. 2001. Tunnel Stability Factor – A new parameter for weak rock tunnelling. Proceedings, 15th Int. Conference on Soil Mechanics and Geotechnical Engineering, Istanbul, Vol. 2, pp. 1403 – 1406

Mihalis, I., Konstantis, S., Anagnostopoulos, A., Vlavianos, G., & Doulis, G. 2009. Tunnel Stability Factor – A new controlling parameter for the face stability conditions of shallow tunnels in weak rock environment. Proc. 17th International Conference on Soil Mechanics and Geotechnical Engineering, Alexandria, 5-9 October, 2009

Mollon, G., Dias, D., & Soubra, A.H. 2009. Probabilistic analysis of the face stability of circular tunnels. Contemporary topics in in situ testing, analysis, and reliability of foundations (GSP 186). Proc. of selected sessions of the 2009 international foundation congress and equipment expo

@Risk 4.5 Advanced risk analysis for spreadsheets, User's Guide, Palisade (www.palisade.com)

Serafeimidis, K., Ramoni, M. & Anagnostou, G. 2007. Analysing the stability of reinforced tunnel faces. In V. Cuellar et al. (eds), ''Geotechnical Engineering in Urban Environments'', Proc. of the 14th Eur. Conf. on Soil Mech. and Geot. Engng., Madrid, Spain, 1079-1084