INTRODUCTION TO A NEW METHOD OF TUNNEL SUPPORT DESIGN: NUMERICAL STUDY WITH THE FINITE ELEMENT METHOD

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ABSTRAK (Indonesia)

Beberapa kelemahan dari Sistim Penyangga batuan berbasiskan Klasifikasi Institut Geoteknik Norwegia (NGI) atau Q adalah sbb: (a) kurang diperhitungkannya arah kekar terhadap permukaan galian terowongan; (b) pengaruh waktu tidak diperhatikan; (c) tidak diperhitungkannya kriteria runtuh getas-daktail (brittle-ductile / Papaliangas) untuk terowongan di kedalaman yang besar dan (d) kurang rasionalnya analisa pengaruh air didalam masa kekar batuan. Paper ini menawarkan suatu metode yang lebih rasional dengan mempertimbangkan pengaruh-pengaruh tersebut diatas berdasarkan rekasyasa numerik: Metode Elemen Hingga Multilaminate untuk masa kekar batuan/joint rock (Zienkiewicz-Pande 1977). Telah dipresentasikan diagram-diagram bunga (Rose Diagrams), tabel-tabel disain dan kurva-kurva yang berguna untuk praktek rekayasa pembuatan terowongan bentuk lingkaran, tanpa / atau dengan 'shotcrecte linning'. Keruntuhan batuan diakibatkan dua pilihan runtuh, yaitu runtuhnya batuan intak (intact rock) atau runtuhnya batuan berkekar (joint rock). Studi numerik terfokus pada terowongan lingkaran, dengan sebuah kedalaman, berbagai tekanan lateral, berbagai arah kekar dan dua kriteria runtuh: kriteria runtuh Mohr-Coulomb dan Papaliangas. Diperkenalkan juga, sebuah cara baru untuk evaluasi pengaruh lapisan shotcrete bagi persyaratan penyangga batuan. Metoda ini menerangkan dan menganalisa interaksi mekanis antara batuan dan penyangga terowongan terhadap waktu. Dipresentasikan juga satu contoh perbandingan metode baru ini terhadap sistim klasifikasi Q. Terdapat dua bagian. Bagian pertama (Sub. Sect. 2 s/d 6) menyangkut analisa elastik FEM dan bagian kedua (Sub. Sect. 7 dst.) menyangkut analisa non-elastik (viscoplastic) FEM, yang berguna bagi instalasi 'shotcrete'.

Kata kunci: tunnel support, Rose Diagrams, Failure Indices, Q-value, Mohr-Coulomb and Papaliangas criterion, interaction diagram, convergence (Penyangga terowongan, Diagram Bunga, Indeks Keruntuhan, Nilai Q, kriteria runtuh Papaliangas dan Mohr-Coulomb, diagram interaksi, konvergensi).

1. INTRODUCTION

Man has built tunnels and caverns for various purposes such as access to mineral resources, tombs, house, shelter, water supply, and drainage system and for transportation. With trains cars and electricity arrived a huge expansion in tunnel construction. For example: the Channel Tunnel (ICE 1993) between England and France. Many kilometers of tunnels are constructed for roads and hydro-electrical projects. Most of these are constructed in rock, which is not competent. Tunnel supports, such as rock bolts, shotcrete lining, steel arches etc. should be required. A breakthrough for rock tunnel engineers appears in 1946. Based on the observation method of the stable excavation profiles of old railway tunnels in the Alps, Terzaghi (1946) proposed to relate rock mass quality with rock load on steel arch supported tunnels. Even though the engineers / geologists have using accumulated past experience in the analysis and design of tunnel supports, these procedures are not readily transferable from one location to another due to the inherent variability of ground conditions and the large number of factors that influence the pressure on the supporting system. Consequently, several empirical methodologies, based on past experience, relating the quality of the rock according to certain classification systems have been proposed. Rock mass classification systems, such as: *RQD* (Deere 1967), *Q-system* (Barton et.al. 1974), *RMR system* (Bieniawski 1990) and *RMi* (Palmstrom 1995) are efforts to classify rock mass properties and condition as a single number. Although they cover a wide range of conditions encountered in the field (Seraphim 1983, Hoek 1995), many of the input parameters are very difficult to measure (Milne 1991). Pande (1995) and Riedmuller (1997) state that in practice, the classification system is inadequate for support determination and stability evaluation in complex geological conditions. In Indonesia, mining and hydro-electrical industries are trend to use Q-system and RMR (Koesnaryo & Rai 1998). They are PLTA and Cirata projects. The Nusantara Tunnel proposed between Sumatra and Java islands, probably will give Indonesian engineers a challenge to create another method (Mangkusubroto 2002). There have also been several attempts to apply numerical methods to tunnel analysis using the following approaches: FEM (Honisch 1988, Lu et.al 1995, Hoek et.al.1995); UDEC (Cundall 1980), FLAC (Brady 1992) and Boundary Element Method. However, a lack of rational modeling of the jointed rock mass (constitutive law); the difficulties in attempts to simulate the detail of rock mass properties and other factors have hampered further developments.

2. PART 1 (ELASTIC ANALYSIS): STABILITY IN ROCK TUNNELS

Failure of the tunnel roof, sides or face takes place when the stresses, which are imposed on the rock mass due to excavation, exceed its strength. The factors which influence the collapse of a tunnel are (a) strength of the jointed rock mass and (b) factors affecting stresses imposed due to excavation. The strength of jointed rock masses is affected by the strength of the intact rock, the presence of joints, mechanical properties of rock joints and also the presence of water (Louhenapessy & Pande 1999, Louhenapessy 2000). The properties or characteristic of rock joint are: dip and orientation, spacing of parallel joint set, number of joint sets and surface roughness. Some factors affecting stresses imposed by excavation on the rock mass are in-situ stress ratio (K_o) and the depth of excavation.

3. CONSTITUTIVE MODELS AND FUNDAMENTAL EQUATIONS

Mohr-Coulomb & Papaliangas models: The Mohr-Coulomb constitutive model for rock joints has been adopted as

$$\tau_j = C_j + \sigma_n \tan\phi \qquad (1)$$

where ϕ and C_j are *friction angle* and *cohesion* respectively for the joint, τ_j is the shear stress on the joint plane and σ_n is the normal stress on the joint plane. The brittle ductile constitutive model for rock joint proposed by Papaliangas is:

$$\tau_j = C_j + \sigma_n \tan(\phi + \psi) \qquad (2)$$

in which,

$$\tan\psi = \tan\psi_o \left\{ \log_{10}\left(\frac{\sigma_{nT}}{\sigma_n}\right) \Big/ \log_{10}\left(\frac{\sigma_{nT}}{\sigma_{no}}\right) \right\} \qquad (3)$$

where σ_{no} is the normal stress on the joint rock plane, ψ is the dilation angle at the instant of peak shear stress, ψ_o is the peak dilation angle under a normal stress (σ_{no}) which causes no (asperity) deformation, σ_{nT} is the effective normal stress which suppress all dilation. The failure criterion for intact rock used is the Mohr-Coulomb criterion as follows,

$$\tau_i = \sigma_{ni} \tan\phi_o + C_o \qquad (4)$$

where σ_{ni} is the normal stress on the failure plane and ϕ_o and C_o are material constants for intact rock.

A general framework for constitutive models for jointed rock masses: The multilaminate framework for developing a constitutive model of jointed rock masses has been discussed in detail in various publications (Zienkiewicz & Pande 1977, Louhenapessy & Pande 1998, Louhenapessy 2003). The elastic constants of a jointed rock mass and rock joint can be determined from large-scale in-situ experiments / large-scale triaxial tests (Natau et.al. 1995). An alternative approach is to derive the elasticity matrix of the jointed rock mass from the constitutive properties of its constituents. The philosophy is to treat jointed rock mass as a composite material with intact rock and rock joints as its constituents. It is to find the normal stiffness and shear stiffness of the rock joint. The elasticity matrix of rock mass, $\mathbf{D_e}^{RM}$, is

$$\mathbf{D_e}^{RM} = \left[\sum_{i=1}^{n} \mathbf{T}\, \mathbf{C}_L^{Joint}\, \mathbf{T}^T + [\mathbf{D_e}^{Intact}]^{-1} \right]^{-1} \qquad (5)$$

where "n" is number of joint sets, **T** is a transformation matrix, **C** is the compliance matrix contains joint stiffness data and $\mathbf{D_e}^{Intact}$ is the conventional elasticity matrix of the intact rock (Pande 90). The recent implementation is for Fault Resistant Design system (Louhenapessy 2008) that inspired by the book by Hori (Muneo Hori 2006).

4. METHODOLOGY OF COMPUTATION OF PRESSURE ON TUNNEL SUPPORTS

The behavior of a jointed rock mass is highly non-linear. Expensive cost of computational (numerical analysis) effort is needed in solving the complex rock-structure interaction problem. Alternatively is to apply, the so-called 'stress path method' in which an estimate is made of the stress path experienced at a few typical points at the tunnel periphery. The stability of these points in the rock mass is considered based on the adopted failure criterion and the support pressure is computed, if required, in such a way that the rock mass is prevented from collapse.

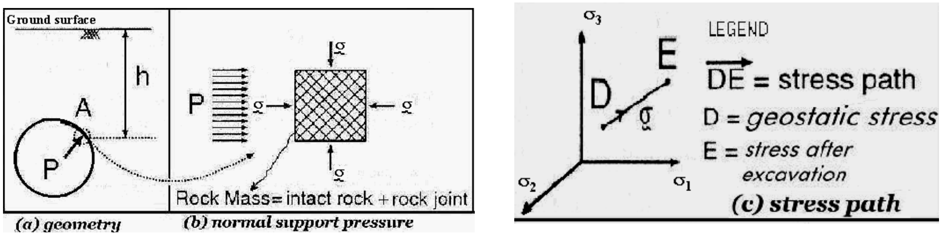
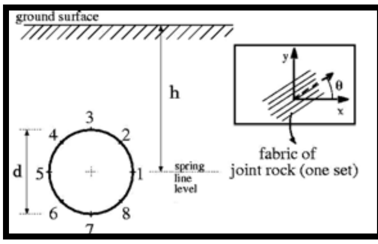


Figure 1. Pressure applied normal to the tunnel periphery at point A to prevent failure of joint rock. (a) geometry, (b) application of normal support pressure to prevent failure under the stress path experienced by point A, (c) stress path experienced by point A in principal stress space.

For example, consider a point such as **A** on the roof of the tunnel (Figure 1) excavated at a certain depth in a jointed rock mass. Before excavation this point experiences geostatic stresses. The stresses at point **A** after excavation which may be in stages can be computed assuming jointed rock mass as an anisotropic multilaminate material having the elasticity matrix given by equation (5) or (6). The deviation of stress from the geostatic condition is readily obtained and gives the stress path to which a rock mass will be subjected at point **A**. This stress path at a point is imposed on rock mass and computation made to judge if failure in any of the following modes is possible: (a) failure of intact rock: The strength parameters of the intact rock are examined and the failure function is checked, and (b) failure of joint sets: The strength parameters are examined and failure in shear or tension is checked. An algorithm, which determines the place of failure onset, i.e.. intact or jointed rock is presented in detail elsewhere (Louhenapessy & Pande 1998, Louhenapessy 2000). If failure is observed in any of the modes, a pressure (p) normal to the periphery of tunnel is computed which would prevent the failure of the rock mass at that point. The procedure is repeated at a number of points on the tunnel's periphery and simple engineering calculations are made to determine the spacing of rock bolts..

5. NUMERICAL EXAMPLES

In this section, analysis of a 12.8 m diameter circular tunnel excavated at 80 m depths (CASE I) in the jointed rock having one sets of joints is presented for the illustration of the methodology of computing support pressures. Here, a two-dimensional idealization is adopted. The notation for the fabric of the rock joints is shown in the inset.



NOTES:
Points **1** *to* **8** *are located on the periphery of the tunnel (e.g. Point* **3** *is at the roof of the tunnel; Point #1 at the right side -spring level- of the tunnel; etc.).* ***d*** *is the diameter of the tunnel,* ***h*** *is the depth of the tunnel and* θ *is dip angle or orientation of joint.*
For CASE I tunnels ***h*** = 80 m

Figure 2. Tunnel Geometry and fabric of rock joint (inset)

Table 1. Material parameters

Intact Rock	Joint Rock
E = 3 x 10^7 kPa and ν = 0.3 Cohession = 28870 kPa $\phi = 30^o$ and ρ = 24.5 kN/m^3	Cn = 1 x 10^{-7} m/kPa and Cs = 2 x 10^{-7} m/kPa Joint Spacing = 1 m and Cohession = 0 Mohr Coulomb $\phi = 20^o$, 30^o and 40^o

Figure 2 depicts the geometry of the problem. The material parameters assumed for illustration are shown in Table 1. Support pressure has been computed for the rock mass having one set of joints at various orientations. The results for one set are presented here. For the calculation of stress paths, elastic finite element analysis is undertaken. Figure 3 shows the typical finite element mesh used for the analysis, which consists of 736 nodes and 224 eight-noded *isoparametric* elements. In view of the approximate nature of the method of calculation, the density of the mesh is not crucial and it is assumed that the mesh shown in Figure 3 gives accurate stress paths for practical purposes. Eight points have been chosen on the circumference of the tunnel (Figure 2) for studying the requirement of support pressure. Three cases of in-situ stress corresponding to K_o = 0.333, 1.00 and 2.00 have been studied. It is noted that cohesion for joints is adopted as zero and the friction angle is varied between 20^o to 40^o.

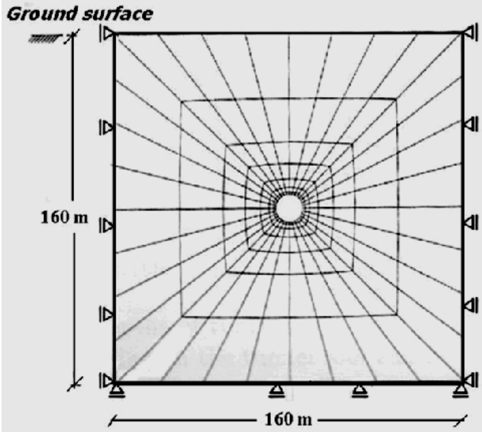
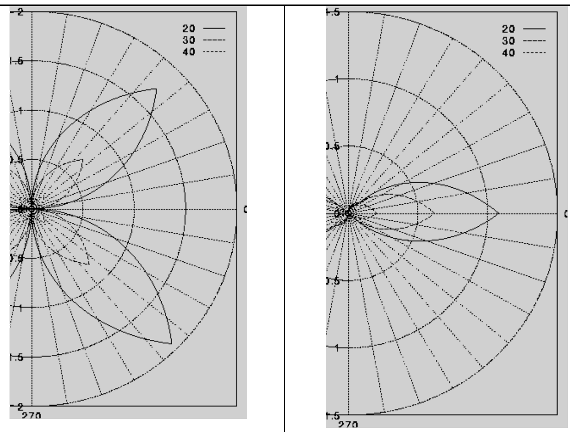


Figure 3 Finite Element mesh CASE-I tunnels



(a) Joint orientation, θ= 0^o *(b) Joint orientation, θ= 45^o*

Figure 4. Normalized Support Pressure (CASE-I)

6. RESULTS

Rose diagrams are useful tools for presenting results of parametric studies of tunnel support pressure analysis. Here the support pressure required at a point on the periphery of the tunnel is plotted as a radial line, the length of which represents the support pressure. Such diagrams are shown in Figures 4a and 4b. The support pressure has been normalized with reference to geostatic stress at the center of the tunnel before excavation $\gamma \mathbf{h}$. Normalized Support Pressure is calculated from the following equation,

$$N = P / (\gamma \mathbf{h}) \tag{6}$$

where N is the normalized support pressure, P is the tunnel support pressure obtained from finite element analysis, γ is the unit weight of rock and **h** is the depth of tunnel. From these rose diagrams, support pressure can be obtained based on the depth of tunnel, joint friction angle ϕ , in situ stress ratio, K_o , and orientation of the joints, θ. The requirement for support pressure varies from point to point on the periphery of the tunnel. Obviously, engineering judgment has to be used and provision should be made for maximum required support pressure in any section.

<u>*Failure Indices:*</u> Zone of Failure Indices (Figure 5a and table in Figure 5b) can be used together in conjunction with the rose diagrams in tunnel support design. The legend of these indices: “0” is <u>no failure</u> zone and “1” is for failure due to <u>intact rock strength exceeded</u>, “2” is for failure due to <u>shear strength of joint rock being exceeded</u> (sliding joint) and “5” is failure due to <u>tensile</u> strength in jointed rock exceeded.

Figure 5a shows the zone of Failure Indices around the tunnel periphery and it shown that appears at the roof (for joint orientation, $\theta=0^o$ and K_o= 0.333). On the other hand, in Figure 5b, tension appears at the side of tunnel (for $\theta=90^o$ and K_o= 2.0). It should be noted that, in most cases –on the tunnel periphery-, failure takes place due to sliding on joints, but there are also situations in which joints open or intact rock fails at very high depth tunnel.

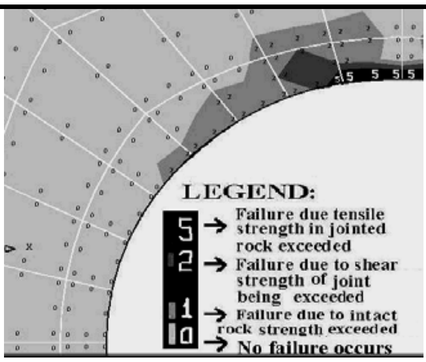
<u>*Comparison with the NGI or Q classification system:*</u> Here we examine a case of the support pressure requirements and compare them with those obtained from the Q classification system. This is a shallow tunnel (CASE-I), with K_o being 0.333, 1.0 and 2.0. The tunnel is in sandstone with one set of joint and intact rock having compressive strength (σ_c) of 100 MPa. The rock mass quality, Q has been calculated based on 6 parameters (Barton *et.al.* 1974). According to Barton, Lien and Lunde (Barton *et.al* 1974), the values of Q and P_{roof} are defined by,

$$Q = \frac{RQD}{J_n}\,\frac{J_r}{J_a}\,\frac{J_w}{SRF} \quad and \quad P_{roof} = 0.667\; J_n^{0.5}\; J_r^{-1}\; Q^{-1/3} \qquad \text{(7a) and (7b)}$$

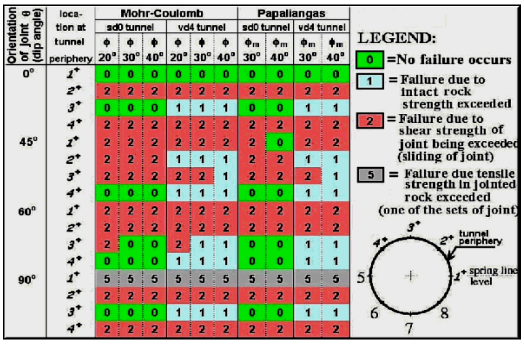
where RQD is the Rock Quality Designation, J_n is the joint set number, J_r is the joint roughness number, J_a is the joint alteration number, J_w is the joint water reduction factor, SRF is the stress reduction factor and P_{roof} is permanent roof support pressure. The following data been assumed for comparison (Palmstrom 1995, Louhenapessy 1998):

- RQD=72 %, J_n= 2 (one joint set), J_r=1.5, J_a=1.0 and J_w=1.0 (dry),
- σ_1 is the maximum principal stresses (from finite element analysis).
- for CASE-I tunnel: $\sigma_1 \approx 1.09$ MPa. $\sigma_c/\sigma_1 \approx 91.5$ (medium stress, SRF = 1.0)
- for CASE-II tunnel: $\sigma_1 \approx 26.67$ MPa. $\sigma_c/\sigma_1 \approx 3.75$ (high stress, SRF = 7.0)

Based on the above parameters: Q = 54.00 and P_{roof} = 16.6 kPa.



a) Zone of Failure Indices around the tunnel. For; θ=0^o, K_o= 0.333, Papaliangas criterion and ϕ_m = 30^o



b) The table above shown failure indices for various orientation of joint, failure criteria, various friction angle & various location on tunnel periphery. This is for insitu stress ration, K_o= 2.

Figure 5. Identification of modes of failure (CASE-I - circular tunnel with one set of joint)

Table 2 shows the tunnel support design of Q system and the proposed method. Roof bolt spacing design using Q system provides only one value for several different parameters and criteria in rock mass. For CASE I, the minimum and maximum bolt spacing design with the proposed method is 262 mm and 7555 mm, which are almost 9 % and 250 % of Q classification system. In general the proposed method gives more extensive information i.e. the zone and extent of area to be rock bolted is indicated. Moreover, they provide a more rational and practical solution as compared to that proposed by any classification system. It is shown that, as expected, the use of Q classification system leads to an over-conservative design in some cases whilst leading to unsafe design in others.

Table 2. CASE -I : Roof Bolt Spacing (Diameter 25 mm)

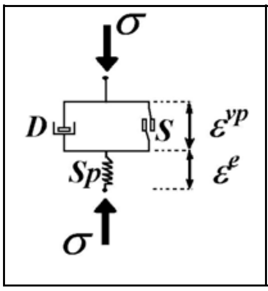
CIRCULAR TUNNEL CASE-I; Depth = 80 m; J_r = 1.5 (JRC=6 or ψ_o=27^o)						
Orien-tation, θ	In-situ Stress Ratio, K_o	Q system (mm)	W_{M-C} (mm) Basic friction angle, ϕ_b		W_{PAP} (mm) Non dilational friction angle, ϕ_m	
			30^o	40^o	30^o	40^o
0^o	0.333	2970	2150	2150	2185	2185
	1.00	2970	No*	No*	No*	No*
	2.00	2970	No*	No*	No*	No*
45^o	0.333	2970	2810	2840	7555	No*
	1.00	2970	400	694	526	No*
	2.00	2970	262	456	292	700
90^o	0.333	2970	No*	No*	No*	No*
	1.00	2970	No*	No*	No*	No*
	2.00	2970	No*	No*	No*	No*

Notes: W_{M-C} = using Mohr-Coulomb criterion; W_{PAP} = using Papaliangas criterion No* = No support required

7. PART 2: ROCK JOINT AS AN ELASTO-VISCOPLASTIC MATERIAL (FOR TUNNEL WITH SHOTCRETE LINING)

The basic theoretical features of an elasto-viscoplastic material are schematically illustrated in Figure 6. The appropriate mathematical / rheological model must describe the following factors: (a) the elastic relation between stress and strain before the onset of plastic deformations, and (b) the yield criterion indicating the stress level at which viscoplastic flow commences. Bingham model has been chosen in multilaminate model. Due to the lack of rheological values (viscosity or fluidity parameter) of most types of rock masses, it has been used in this study the simplest possible curve fitting for the purpose of back analyzing measured values (Louhenapessy 2003).

When a tunnel is driven through a rock mass, its stability is disturbed, causing an increase in the roof or wall displacements with time for several weeks or months after excavation (Panet & Guenot 1982, Barla 1999). Measuring the new distance between two points on the tunnel periphery (say points #1 and #5 of Figure 2) known as convergence measurements. The displacements will reduce the distance between the two points. Therefore, the time-convergence relationship and rock rheology play important roles in modern tunnel design. A shotcrete lining is a type of tunnel surface support that includes cast concrete lining. The characteristics of shotcrete tunnel support are (a) load transfer from rock mass to the shotcrete starts in the early stages of shotcrete installation, (b) shotcrete with high early strength has been used and (c) the shape of shotcrete lining follows the contour of tunnel surfaces. Q classification system, lack time-dependency factors (Bentmark 1998) whilst others have the time dependency factors but lack scientific rationale (Terzaghi 1946 and Bieniawski 1990). The new procedure is a simulation of the shotcrete installation that takes into account the relaxation of the rock (Louhenapessy 2000, Louhenapessy 2003).



LEGEND:
D = Dashpot,
S = Slider and
Sp = Spring Strains :
ε^e = elastic strains, and ε^p = plastic strains.

Stresses σ imposed stresses on the rock mass

Figure 6 Rheological analogue of elastoviscoplastic (Bingham Model)

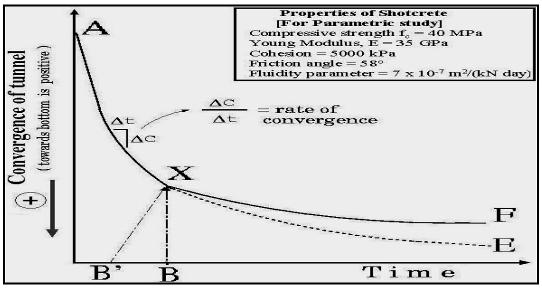


Figure 7. The method of describing and analyzing the mechanical interaction between rock, tunnel support with time.

8. ROCK-SUPPORT (SHOTCRETE) INTERACTION ANALYSIS

The representative curve method of describing and analyzing the mechanical interaction between rock and tunnel support with time, has been introduced in a simplified schematic version presented in Figure 7. The optimum design will be reached when the convergence of tunnel and shotcrete reduces to a minimum, i.e. where the graph AXF (Figure 7) extends parallel to the "Time" axis. This means that the pressure required to limit the rate of convergence is counteracted by the pressure available from the shotcrete.

Point A is the start of the tunnel convergence. At point X (Figure 7) the rock and shotcrete will behave as a combined system, as one material body. The installation of shotcrete can be divided as follows:

- When the installation of shotcrete begins at point X and using the assumption mentioned previously, the line BX becomes the support response curve. Note that the line is vertical, which means that $\Delta t = 0$ (shotcrete with high early strength has been used).
- Line B'X is where shotcrete is assumed to gain the strength several days after the installation. Note that, in this case $\Delta t \neq 0$. This is beyond the scope of this paper.
- The rest of the remarks on Figure 7 are: AXE = convergence curve without shotcrete and XF = convergence curve after installation of shotcrete.

In this present work, the FEM of the tunnel without shotcrete is first analyzed using an elasto-visco-plastic formulation for the rock mass. This predicts the time-dependent history of convergence. At a certain stage of tunnel analysis, the shotcrete lining is placed instantaneously and the revised history of convergence with lining is computed.

9. IMPORTANT ASSUMPTIONS

Plane strain approximation in conjunction with a finite element "multilaminate" model, has been used in the analysis of tunnel and tunnel lining (shotcrete). The construction work is simulated by one step of excavation only. For the sake of the simplicity of the FEM model, this paper assumes that shotcrete attains its full strength during the early stages. The shotcrete yield criterion is Mohr-Coulomb, with known values of c and ϕ obtained from interpreted compressive strength, f_c, and tensile strength, f_{ct}, of concrete (Louhenapessy 2000). The primary lining shotcrete has a specified characteristic compressive strength of 40 MN/m^2 and tensile strength of 2.5 MPa. The relation between the modulus of elasticity and the compressive strength has a great influence on the shotcrete (Chan 1994), and it can be expressed as,

$$E_c = 3.86\, f_c^{0.86} \tag{8}$$

The numerical model for the tunnel with shotcrete lining is applied to a tunnel of 12.8 m diameter and 80 m depth (CASE I, Figure 3). Various in-situ stress ratios, K_o and rock joint friction angles, ϕ, have been used: K_o = 0.333 and 2.0. The condition of the joint orientation is the primary objective of the present investigation, therefore various orientations of joint have been used. The parameters adopted in the design of the tunnel lining are shown in Table 1. Figure 8 shows zoom of the typical mesh used for the analysis, which consists of 832 nodes and 236 elements.

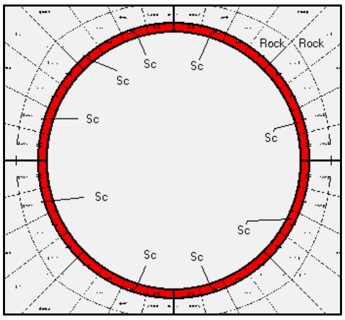


Figure 8. Zoom of shotcrete Finite Element mesh near the periphery of CASE-I tunnel (▬ sc = element mesh for shotcrete, **Rock** = element mesh for rock)

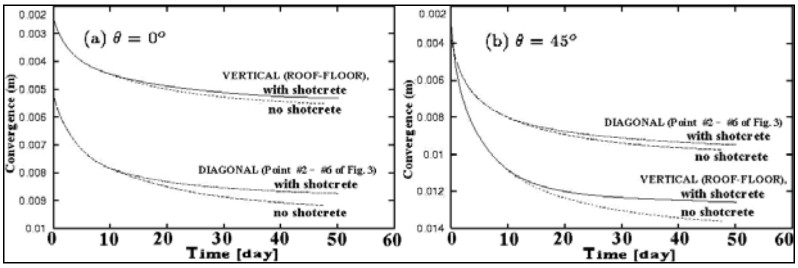


Figure 9. Convergence of "CASE-I" tunnel with and without shotcrete, with Ko = 0.333, one set of joint, friction angle $\phi = 20^o$ [using Mohr-Coulomb criterion] and Time installation is 50 Time Step (or 10 days).

10. PARAMETRIC STUDY AND RESULTS

Parametric studies: Parametric studies show different times for installation of shotcrete and the influence of shotcrete lining on the requirement of normalized support pressure on the tunnel periphery. Based on the convergence measurements analysis (the use of visco-plasticity) made in the Frejus Tunnel (Louhenapessy 2000) it is assumed that the fluidity parameter, γ is 7 x 10^{-7} m^2/(kN day).

Results: Some of the results of this numerical study are shown in Figures 9 and 10. They indicate that the influence of both joint orientation, joint rock friction angle, time of installation of shotcrete and different failure criteria plays an important role in the design of tunnel. This is lead to an important role in the tunnel support design.

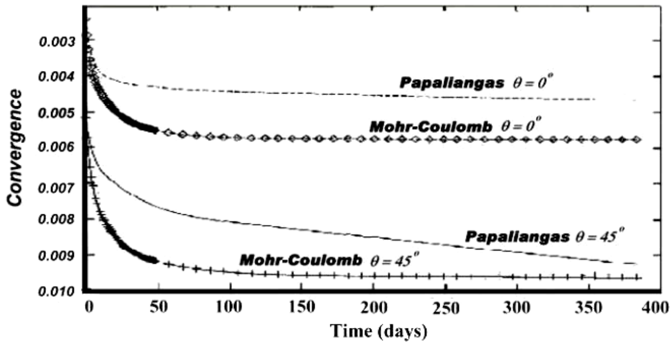


Figure 10. Convergence of tunnel: Associate Flow, Using friction angle $\phi = 30^o$ [Mohr-Coulomb failure criterion] and $\phi_m = 30^o$ and JRC = 6 or $\psi = 27^o$ [Papaliangas failure criterion].

11. CONCLUSION

In Part 1: Analysis and design of tunnel support system is a complex problem of rock structure analysis. In this paper a rational but practical method of computing support pressure has been suggested. It is based on the 'stress path' method of analysis. The stress path at a number of point on the periphery of the tunnel is computed using an elastic finite element method. A multilaminate theory is used to compute the support pressure which would prevent the collapse of the rock mass. The methodology is explained by a set of rose diagrams and tables of failure indices. It is proposed that the engineers should develop similar diagrams for the tunnel based on actual laboratory / field data. These design charts can be readily read for any situation during construction. The methods of excavation of support pressure based on a classification system lack rationale and should be used with caution.

In Part 2: The application of the new method describing and analyzing mechanical interaction between rock and tunnel support (in this case tunnel lining/shotcrete) with time has been performed. Parametric study of convergence analysis indicates that the influence of both joint orientation, joint rock friction angle, time of installation of shotcrete and different failure criteria plays an important role in design of tunnel.

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