Design of pressure tunnels using a finite element model

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In this study, a two-dimensional plane strain finite element model is used for the prediction of mechanical and hydraulic behaviour of pre-stressed concrete-lined pressure tunnels. Results obtained from numerical models are presented and compared with the one calculated using analytical solutions.

This paper is part of the study on the coupled stress-seepage numerical design of concrete-lined pressure tunnels and is particularly concentrated on the predicted behaviour of uncracked concrete-lined pressure tunnels based on finite element model (FEM) analysis. The study covers the modelling of tunnel excavation, installation of support systems, pre-stressing of concrete lining and the loading of internal water pressure. A circular concrete-lined tunnel is modelled. This design was selected as being the most suitable cross-section for pressure tunnels in a deep homogeneous isotropic rock mass subject to a constant in-situ compressive stress. The jointed rock mass properties are defined based on the Hoek-Brown failure criterion.

The analysis begins with the definition of normal in-situ principal stresses along the longitudinal axis of excavation to predict excavation-induced stresses and deformations. As a result of the excavation works, the stress levels surrounding the site are changed. In this case, the behaviour of rock mass may no longer remain elastic and therefore the zone where the plasticity occurs has to be predicted.

The mechanical performance of the rock mass in the elastic-plastic condition was studied using the FEM. An adequate support system, taking into account the stress release in front of the excavation face, was designed to anticipate the initiation of plastic failure and instability problems of the tunnel. Once the equilibrium condition around the supported tunnel is reached, a final lining can be installed on the support element and a high-pressure cement grout injected through the radial boreholes. This passive pre-stressing technique produces enough compressive stress in the final lining to suppress crack openings.

1. Background

Over the last 25 years, the design of pressure tunnels at hydropower schemes has developed considerably. The need for a more economical design has consequences for the choice of improving the bearing capacity of concrete-lined pressure tunnels. One of the techniques available to improve their applicability is by pre-stressing the final lining with a high-pressure cement grout.

Since it first appeared in Seeber [1985[1]], the load-line diagram has been widely used to determine the bearing capacity of pre-stressed concrete-lined pressure tunnels. This diagram remains widely referred to and the most recent edition is found in Vigl and Gerstner [2009[2]]. As well as economic benefits, another reason for the popularity of this technique is that a tight contact between the lining and the rock mass can be achieved, and thus the load from the lining to the rock mass can be transferred continuously.

Principally, the level of grout pressure injected through radial boreholes depends on the rock mass strength, which should not exceed the smallest main axial stress in the rock mass so as to avoid the hydro-jacking and/or hydro-fracturing of surrounding rock mass. The tangential compressive strains induced by pre-stressing are estimated using the load-line diagram, which was developed based on the linear elastic relationship between the rock mass and concrete lining. However, rock masses do not have a linear behaviour. In reality, the rock mass failure is controlled by numerous joint surfaces and therefore, the non-linear Hoek-Brown criterion is applied in this study to estimate the performance of the structural behaviour of pressure tunnels.

Because of the applicability of the FEM in dealing with rock mass problems, among others, non-linear deformability, material inhomogeneity and complex boundary conditions, this method has been widely applied. In this paper, the FEM is used and an attempt has been made to describe the mechanical and hydraulic behaviour of concrete-lined pressure tunnels.

The objective of the study is to provide an overview of a practical application of FEM for the design of pre-stressed concrete-lined pressure tunnels which are divided in three parts consecutively, dedicated to tunnel excavation and support installation, pre-stressing of the final lining, and activation of internal water pressure. Only a few examples of the implementation of FEM for the design of pre-stressed pressure tunnels are given in the literature, among others are found in Stematiu *et al.* [1982[3]] and Marence [1996[4]]. While Stematiu *et al.* assumed the rock mass to behave as elastic material, Marence considered the plastic behaviour of the rock mass defined by Mohr-Coulomb parameters.

It has been recognized that the Mohr-Coulomb law is not suitable for a rock mass. The predicted behaviour of rock mass as a result of excavation is therefore based on the non-linear yield function given by Hoek-Brown rock plasticity. Using the convergence confinement method, the appropriate location of the installation of the support lining, taking into account the three-dimensional effect of excavation, can be determined. The redistribution of stresses and deformations after the installation of the support lining is predicted using a FEM. Then, the modelling of pressure tunnels progresses to the installation of the final lining, pre-stressing of the final lining with high-pressure cement grout and loading of the internal water pressure. The pre-stressing is modelled by applying a uniform load at the support and final lining interface.

For model validation, results of excavation-induced stresses and deformations as well as their redistribution after the installation of support obtained from FEM are compared with those calculated using the elastic-plastic solution as described in Carranza-Torres [2004[5]]. The seepage pressure distribution is compared with the analytical solution proposed by Schleiss [1986[6]] and the prediction of integrated bearing capacity of the system based on the stress-seepage coupling mechanism, is compared with the one proposed by Simanjuntak *et al*. [2012[7]].

2. The Hoek-Brown rock plasticity model

In cases of rock masses that exhibit non-significant anisotropy in strength and deformability, the assumptions of isotropic behaviour are reasonable. However, non-linear criteria have to be used, since the strength of the rock mass does not increase linearly with the level of stresses. Since being first introduced in 1980, the failure of a jointed rock mass in response to induced stresses is usually described by using the Hoek-Brown failure criterion. Consequently, the definition of rock mass parameters used in the numerical model is determined based on the Hoek-Brown criterion. Some relevant examples can be found in Carranza-Torres and Fairhurst [1999[8]], Carranza-Torres [2004[5]], Sharan [2005[9]] and Clausen and Damkilde [2008[10]].

Since the full mathematical treatise of the Hoek-Brown criterion is already presented in Hoek *et al* [2002[11]], this paper concentrates on the plastic potential that can be distinguished for non-associated and associated material behaviour. The plastic potential, g, is defined as:

$$g = \sigma_1 - \sigma_3 - \sigma_{ci}\left(m_g \frac{\sigma_3}{\sigma_{ci}} + s_g\right)^{a_g} = 0 \quad \text{... (1)}$$

The plastic potential is needed to control volume change through plastic dilation. The rate of dilation is controlled by the parameter m_g, in which with relation to the dilation angle ψ, this parameter is calculated using:

$$1 + m_g = \frac{1 + \sin\psi}{1 - \sin\psi} \quad \text{... (2)}$$

Practical examples of the implementation of the Hoek-Brown criterion considering the associated and non-associated flow rule conditions are available in Carranza-Torres and Fairhurst [1999[8]] and Clausen and Damkilde [2008[10]]. However, in cases of plane strain in isotropic rocks with failure criteria independent of the main intermediate stress, the use of plastic non-associated flow rule is more appropriate [Hoek and Brown, 1997[12]; Serrano *et al*., 2011[13]; Wang, 1996[14]], meaning that the rock mass undergoes no change in volume during plastic deformation. Another reason to disregard the associated flow rule is that its application overestimates the degree of plastic dilation and thus plastic dissipation [Wan, 1992[15]].

3. Excavation-induced stress and deformation

As a result of the tunnel excavation, the equilibrium state of initial stress is disturbed and the first deformations occur. The rock mass surrounding the excavation may not remain elastic anymore and may deform non-elastically. To define the stress and deformation associated plasticity, the Hoek-Brown criterion and the non-associated flow rule material behaviour are used. Studies on the elastic-ideal plastic analysis of stresses and displacements around a circular excavation in the Hoek-Brown media are available in the literature and in this paper. The results obtained from numerical models for the calculation of excavation-induced stress and deformation in the elastic region are compared with the one proposed by Sharan [2005[9]]. For the plastic region, the series of formulae described in Carranza-Torres [2004[5]] and Carranza-Torres and Fairhurst [2000[16]] are precise.

4. Convergence confinement method

Since the state of stresses and displacements around the tunnel excavation is actually three dimensional, three-dimensional methods should be used for the proper analysis of distribution stresses and displacements during the tunnelling processes. If one of the principal components of the in-situ stress is acting parallel to the longitudinal axis of the excavation, this complicated analysis is usually replaced by cross-sectional plane-strain analysis [Unlu and Gercek, 2003[17]].

Methods based on the two-dimensional plane strain analysis have been developed in the past on establishing the relative position of the tunnel face and the sections under consideration using certain approximations which takes into account the stress release in front of the face of excavation, among others, convergence-confinement method [Panet and Guenot, 1982[18]], progressive softening method [Swoboda *et al*., 1993[19]] and hypothetical modulus of elasticity soft lining method [Karakus and Fowell, 2003[20]]. These approximations reflect the deformations that occur between the excavated area and the application of the support system.

Nevertheless, a method to predict the support loads must consider not only the stress release occurring before the installation of the support system, but also the plastic behaviour of the ground, as well as that of the elastic ground [Kim and Eisenstein, 2006[21]]. The elastic-plastic behaviour of the rock mass is incorporated in the convergence confinement method and therefore this method is selected in this study to calculate the load imposed to the support lining.

Several studies have been done in the past on the correct approximation of longitudinal deformation profile. When the measured data are available, an empirical best-fit relationship as suggested by Carranza-Torres and Fairhurst [2000[16]] is adequate. In cases where measured data cannot be realised, three-dimensional numerical models can be used. Formulae developed by Vlachopoulos and Diederichs [2009[22]] is recommended in this paper so as to validate the results from numerical modelling given that the influence of the development of a plastic zone is taken into account in the prediction of deformation after shotcrete installation. The best-fit relationship between radial displacement and the points located ahead and behind the face are calculated consecutively using:

$$\frac{u_r}{u_r^{\max}} = \frac{e^{-0.15\,(R_{pl}/R)}}{3} \cdot e^{(x/R)} \qquad \text{... (3)}$$

$$\frac{u_r}{u_r^{\max}} = 1 - \left[\left(1 - \frac{e^{-0.15\,(R_{pl}/R)}}{3}\right) e^{-1.5\,(x/R_{pl})}\right] \qquad \text{... (4)}$$

with R and R_{pl} as the radius of excavation and the elastic-plastic interface respectively.

Once the longitudinal deformation profile has been established, another question of maximum tolerable convergence arises, since it is impossible, except when very special techniques are used, to install the shotcrete directly at the tunnel face. In some cases, the critical convergence of up to 5 per cent may be used; nevertheless, as recommended by Hoek [2000[23]], a critical value of 1 per cent should not be exceeded since it would cause distress.

5. Grouted zone

If the rock mass in the vicinity of pressure tunnels has a relatively high permeability, the rate of leakage flowing out of the tunnel has to be minimized to avoid possible hydro-jacking of the surrounding rock mass, washing out the joint fillings and associated hazards, such as landslide, environmental impacts, flooding of the adjacent powerhouse or even the collapse of the tunnel. If the safety of the tunnel is not put at risk, the rate of water leakage, q, in the order of 2 l/s/km/bar is still tolerable [Marence, 2008[24]]. The excessive leakage imposed by the internal water pressure can be prevented by application of consolidation grouting. The necessary radius of rock mass which needs to be grouted, r_g, can be calculated using Eq. (6) given in Schleiss [1986[25]].

Table 1A: Tunnel geometry			
Parameters	Symbol	Value	Unit
Internal radius of final lining	r_i	1.6	m
External radius of final lining	r_a	1.9	m
Shotcrete thickness	t_s	0.1	m
Radius of excavation	R	2.0	m
Radius of grouted zone	r_g	3.0	m
Table 1B: Rock mass properties			
Parameters	Symbol	Value	Unit
Uniaxial compressive strength (intact)	σ_{ci}	30	MPa
Hoek-Brown constant	m_b	1.677	-
Hoek-Brown constant	s	0.00387	-
Dilation angle	ψ	0	o
Modulus of deformation	E_{rm}	5.5	GPa
Poisson's ratio	ν	0.25	-
Table 1C: Concrete properties			
Parameters	Symbol	Value	Unit
Unconfined compressive strength	σ_{cc}	35	MPa
Young's modulus	E_c	30	GPa
Poisson's ratio	ν_c	0.25	-

6. Integrated bearing capacity of pre-stressed concrete-lined pressure tunnels

The bearing capacity of pressure tunnels can be improved by creating a certain pre-stress in the lining, through the injection of high pressure cement grout into the circumferential gap between the shotcrete and final lining. The calculation of pre-stress-induced strain at the intrados of concrete lining assuming the tunnel is impervious can be found in Seeber [1985[1]].

In reality, a concrete lining cannot be made totally impervious. The bearing capacity of pressure tunnels therefore has to take into account the effect of seepage pressure. Using the double thick-walled porous cylinder theory, the seepage-induced stress at the intrados of concrete lining can be predicted. A full mathematical treatise of the method has been presented in Schleiss [1986[6]].

Recent studies on the mechanical and hydraulic effects of grouting towards the improvement of bearing capacity of pre-stressed concrete-lined pressure tunnels has been carried out by Simanjuntak *et al.* [2012[7]]. The integrated bearing capacity of pre-stressed concrete-lined pressure tunnels is calculated using the coupled stress-seepage analysis expressed in terms of the residual tangential compressive strain at the intrados of concrete lining, and this should not fall beyond zero for the most unfavourable loading combinations.

$$\varepsilon^{t}_{c,P_p} + \varepsilon^{t}_{c,P_i} \le 0 \qquad \text{... (5)}$$

$\varepsilon^{t}_{c,Pp}$ and $\varepsilon^{t}_{c,Pi}$ are respectively pre-stress-induced strain and seepage-induced strain at the intrados of concrete lining.

7. Pressure tunnels modelling

This section is intended to present an overview of the modelling of pre-stressed concrete-lined pressure tunnels simulated in three consecutive phases: tunnel excavation and installation of support lining; installation of final lining and pre-stressing; and, the activation of the loading of internal water pressure inside the tunnel.

The calculation of stresses and deformations from each phase is predicted using commercial finite element software DIANA [2012[26]]. It is a deep tunnel and assumed to be located above the groundwater level in an isotropic homogenous rock mass subjected to a constant hydrostatic in-situ stress. In DIANA, two types of analysis are coupled, namely the structural non-linear and steady-state groundwater flow analysis. The general description of the modelling procedure and assumptions used for each phase is presented in the following sub-sections.

7.1 Model set-up

For computation, an opening with a 2 m radius of excavation in an infinite elastic-plastic rock mass was used in this study. The data for intact rock properties were adopted from the one given in Carranza-Torres and Fairhurst [1999[8]]. For the determination of rock mass parameters satisfying the Hoek-Brown criterion, formulae described in Hoek *et al* [2002[11]] were applied. The thickness of support and final lining are taken consecutively as 10 and 30 cm. The thickness of the grouted rock mass is 1 m.

Since the load and geometry of the pressure tunnels are symmetrical, only half of the model geometry is considered to represent the whole tunnel. The model domain is made free to move in the radial direction, but not in the longitudinal direction. A two-dimensional plane strain condition is used meaning that only the tangential and radial strain components are considered. To avoid using an excessively large model, the minimum required model boundary suggested by Ruse [2003[27]] was applied. A highly refined mesh based quadrilateral elements was set (Fig. 1b).

7.2 Loading steps

7.2.1 Phase 0 - Stress initialisation

This step is aimed to simulate the primary state of stress in the rock mass before the excavation. The Hoek-Brown rock mass is assumed to be in elastic-ideal plastic condition. The load was set by introducing a constant in-situ compression stress of 30 MPa. The horizontal-to-vertical stress coefficient was taken as 1.

7.2.2 Phase 1 - Tunnel excavation and support lining installation

Two cases are considered in this phase: modelling of the tunnel excavation without and with a shotcrete lining. The compulsory partitions for these two cases are summarized as follows.

Principally, for modelling of tunnel excavation, two boundary conditions are applied. The first boundary condition, which is located at the outside model domain gives an upper value of the radial stresses, while the other, located at the tunnel wall, represents the support pressure. For the modelling of tunnel excavation with a shotcrete lining, the second boundary condition is introduced to the model by activating the element representing the shotcrete lining. For the rock mass, the non-linear analysis according to the Hoek-Brown yield criterion was performed and the yield value as given in Table 1B [Carranza-Torres and Fairhurst, 1999[8]] was applied.

The material model for shotcrete was designed according to the concrete model, that means, it is capable of handling combined tension and compression. Here, the biaxial stress state in the concrete can be modelled by a combination of the yield criteria of Rankine and Von Mises. The relevant concrete properties, based on Eurocode 2 and presented in Table 1C, were used as an example.

7.2.3 Phase 2 - Installation of final lining and pre-stressing

While the shotcrete lining absorbs some of the stresses and deformations resulting from tunnel excavation, the final lining is designed to take the loading imposed by the internal water pressure. The final lining is concreted directly on the shotcrete lining after the equilibrium condition has been reached. As a result of the concreting processes, self-weight of lining, creep and shrinkage effects, the final lining detaches from the shotcrete developing the circumferential gap between them. This occurs especially at the tunnel crown. In practice, this gap is closed with contact grouting, using low pressure (of up to 5 bar), to achieve full contact in the system.

The modelling of pressure tunnels proposed in this paper is oriented towards the concept of the compatibility condition meaning that full contact between shotcrete and final lining is achieved after the contact grouting application. Therefore, there are no tensile strains and

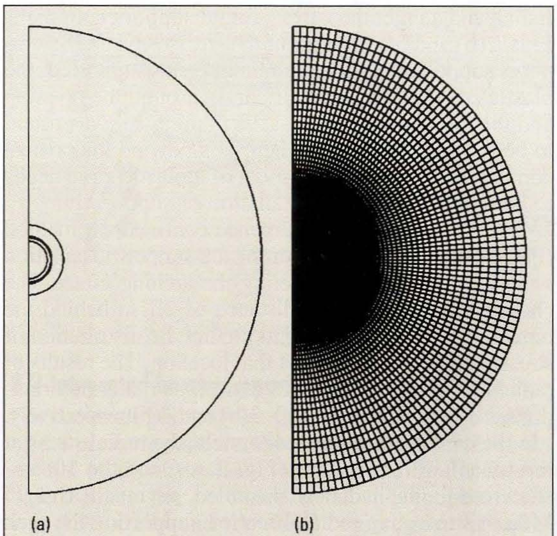

Fig. 1. Model geometry and mesh.

tensile stresses in the shotcrete-final lining interface. The pre-stressing of final lining was modelled by setting a constant uniform pre-stress load at the shotcrete-final lining interface. The pre-stress grout of 10 bar was used as an example and again, the combination of yield criteria of Rankine and Von Mises was applied.

7.2.4 Phase 3 - Loading of internal water pressure

For this phase, two permeable boundaries are set up, i.e. at the intrados of concrete lining and at the outside of model domain. Some quite moderate improvements by consolidation grouting are assumed to occur to the hydraulic parameter of the rock mass. As an example, the permeability of the concrete lining and shotcrete, grouted zone, and ungrouted rock mass, were taken as 10^{-8}, 5×10^{-8} and 10^{-6} m/s, respectively. The internal water pressure of 14 bar is applied as the maximum loading of the internal water pressure.

While the permeable boundary at the outer model domain characterizes the hydrostatic head imposed by the groundwater level, the permeable boundary at the intrados of concrete lining represents the hydrostatic head imposed by the internal water pressure. Since the groundwater level is not present, the hydrostatic head at the outer permeable boundary was set to zero. The other hydrostatic head representing the internal water pressure at the intrados of concrete lining was set to 14 bar.

The distribution of stresses and deformations induced by the internal water pressure was simulated based on non-linear structural analysis. 14 bar-internal-water-pressure was introduced to the model by setting the uniform pressure load of 14 bar at the intrados of the final lining for the simulation of seepage-induced stress, the steady state groundwater flow analysis was applied.

8. Results and discussion

While for Phase 1 and Phase 2 only the structural non-linear analysis was performed, the structural non-linear analysis of Phase 3 was coupled with the groundwater flow stress analysis. For each phase, successive iterations were performed until the equilibrium in forces and deformation was achieved.

8.1 Modelling results

8.1.1 Phase 1 - Tunnel excavation and support lining installation

While Figs. 2(a) and 3(a) represent the distribution of radial and tangential stress for unsupported tunnels, Figs. 2(b) and 3(b) represent the stresses when the tunnel is supported. When the tunnel is not supported, the plastic zone develops significantly around the opening and the total deformation at the tunnel wall was found to be 8.01 cm (Fig. 4a) which falls beyond the critical convergence. The development of a plastic zone needs to be reduced with the installation of support lining.

With the aid of the convergence confinement method (Fig. 5), the optimal location of the support installation was determined by considering the arching effect. The chosen location was at a distance of 2.1 m behind the tunnel face. FEM was used to predict the distribution of stresses and deformations at that location. The results of radial and tangential stresses as well as radial deformations are shown in Figs. 2(b), 3(b) and 4(b) respectively.

In the case of unsupported tunnels, the radial stress at the tunnel wall was zero (Fig. 2a). Once the 10 cm-shotcrete-lining had been installed, as much as 1.95 MPa of stress, in a radial direction supporting the rock mass, acted at the boundary between the rock mass and shotcrete lining. Accordingly, the radial displacement at the tunnel wall decreased to 4.17 cm. The radial displacement in equilibrium is within the acceptable limit of critical convergence. The classic jump for both tangential and radial stresses occurring at the elastic-plastic boundary was predicted to take place at a 4.6 m radius. Fig. 6 shows a comparison of results for both unsupported and supported tunnels. These figures show agreement between the results of numerical and analytical models.

According to the elasto-plasticity and compatibility condition, the excavation-induced stress and deformation at the tunnel wall were transferred continuously to the shotcrete once it was installed. The radial stress at the intrados of shotcrete lining was predicted to be 0.58 MPa with the corresponding radial displacement of 4.4 cm.

8.1.2 Phase 2 - Installation of final lining and injection of high pressure grout

In this phase, the 30 cm-final-lining was installed on the shotcrete lining and radial stress and deformation acting at the intrados of shotcrete lining was transferred continuously to the extrados of the final lining. Since the equilibrium condition was reached and there was no additional load from the rock mass, the radial stress and displacement imposed by the shotcrete diminished to zero on the intrados of the final lining. After that, a uniform pre-stressing load of 1 MPa was applied at the shotcrete-final lining interface resulting in as much as 1.15 MPa of compressive stress in a radial direction at the extrados of final lining (Fig. 7a).

8.1.3 Phase 3 - Loading of internal water pressure

As shown in Fig. 8a, as little as 0.14 Pa of tensile stress in a radial direction was found at the intrados of concrete lining as a result of the loading of 14 bar-internal-water-pressure. However, this tensile stress will not develop cracks in the final lining, since the tangential stress at the intrados of concrete lining still remained in a compressive state. During the operation

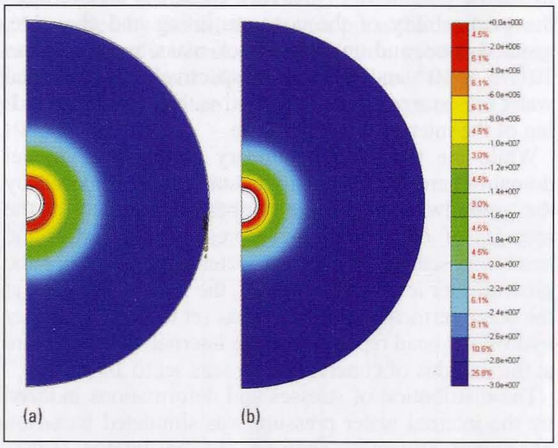

Fig. 2. Distribution of radial stress.

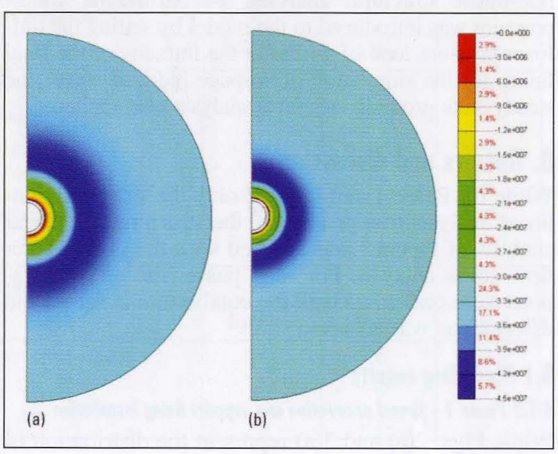

Fig. 3. Distribution of tangential stress.

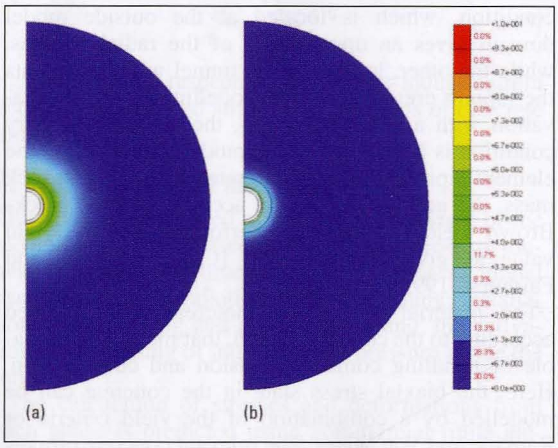

Fig. 4. Distribution of radial deformation.

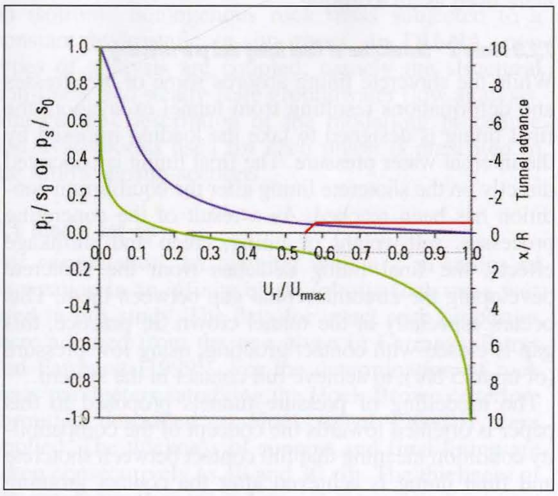

Fig. 5. Predicted location of shotcrete installation.

of pressure tunnels, the tangential stress acting at the intrados of final lining was predicted to be 1.59 MPa (Fig. 8b).

Furthermore, seepage pressures propagate through the permeable final lining, the shotcrete and the surrounding rock mass and their magnitudes depend on the groundwater level and permeability of the system, that is, the concrete lining, shotcrete, grouted zone and the rock mass. For the case where there is no groundwater level present above the tunnel, the distribution of seepage pressures are exclusively controlled by the permeability properties of the system. Then, the saturated zone is developed because water leaks out of the tunnel and its distribution around the tunnel is evident by the characteristic line as illustrated in Fig. 9.

The distribution of seepage pressure around the pressure tunnel is shown in Fig. 10. From the analysis, the predicted seepage pressures at the intrados and extrados of the grouted zone were found to be 4.33 and 0.82 bar, respectively. When computed using the double thick-walled porous cylinder theory [Schleiss, 1986[6]], the corresponding seepage pressure were 4.24 and 0.7 bar. Again, results obtained using numerical models were in agreement with analytical solutions.

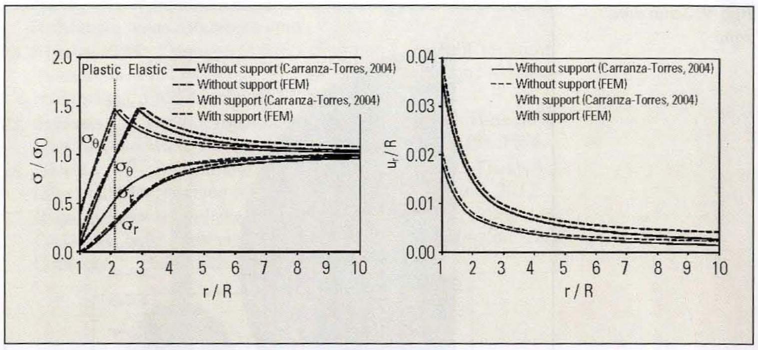

Fig. 6. Comparison of analytical and numerical results for tunnel excavation without and with support.

8.2 Integrated bearing capacity of pressure tunnels

For the evaluation of the serviceability and reliability of the pressure tunnels, the magnitude of residual tangential strain at the final loading condition is investigated. Since the predicted residual tangential strain was 4.98 x 10^{-5} and remained compressive, it can be concluded that the pre-designed value of internal water pressure, which is 14 bar, is still within the safe margin of tunnel design. If the coupling stress-seepage mechanism proposed by Simanjuntak *et al* [2012[7]] is used, the residual tangential compressive strain is calculated to be 3.76 × 10^{-5}.

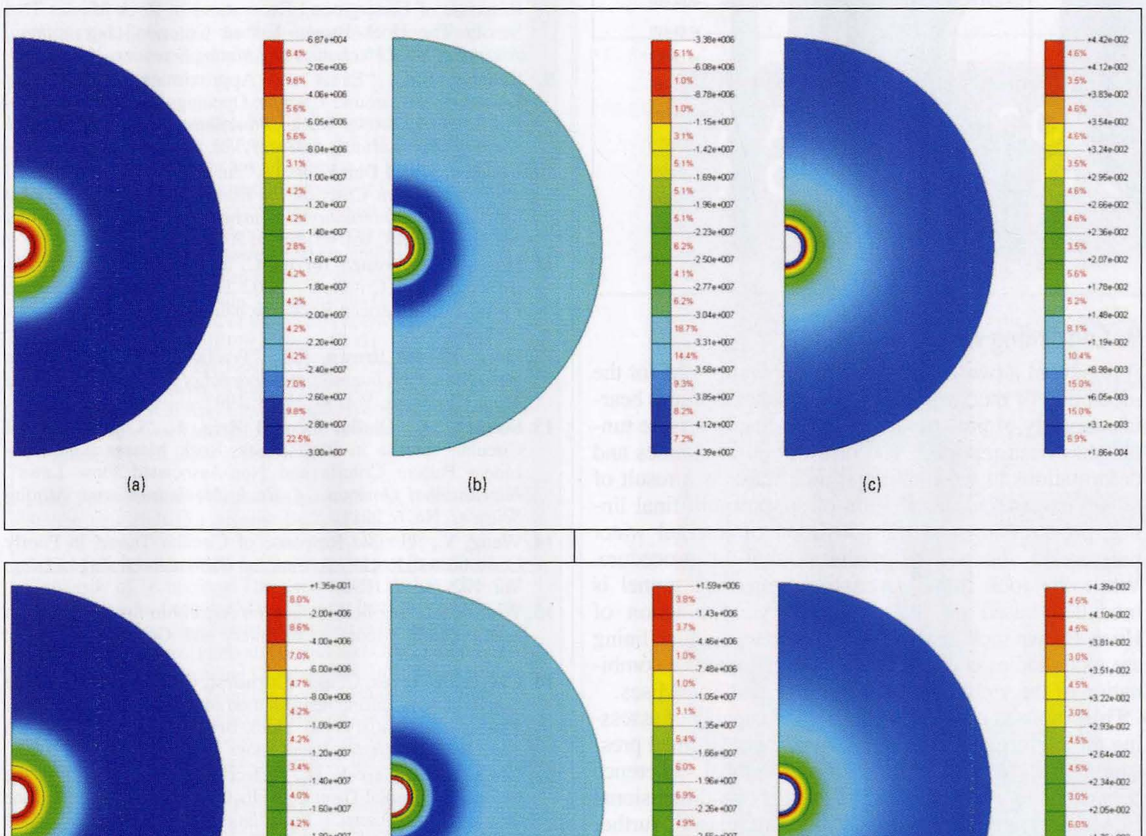

Fig. 7. Radial stress, tangential stress and radial displacement after prestressing of final lining.

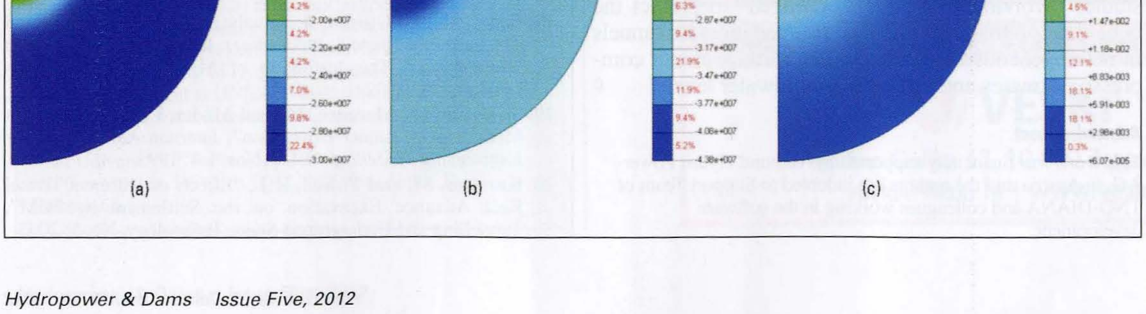

Fig. 8. Radial stress, tangential stress and radial displacement after loading of internal water pressure.

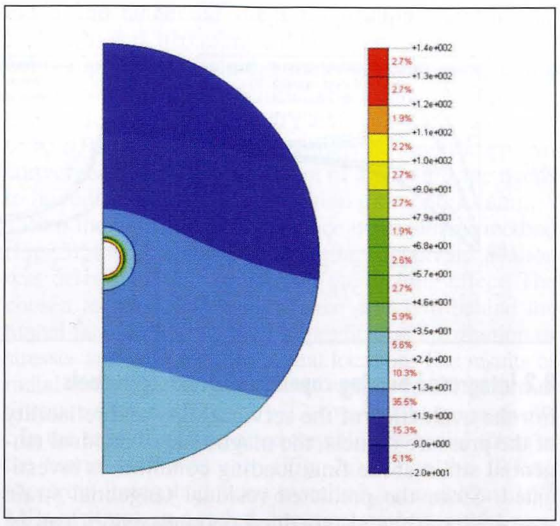

Fig. 9. Saturated zone.

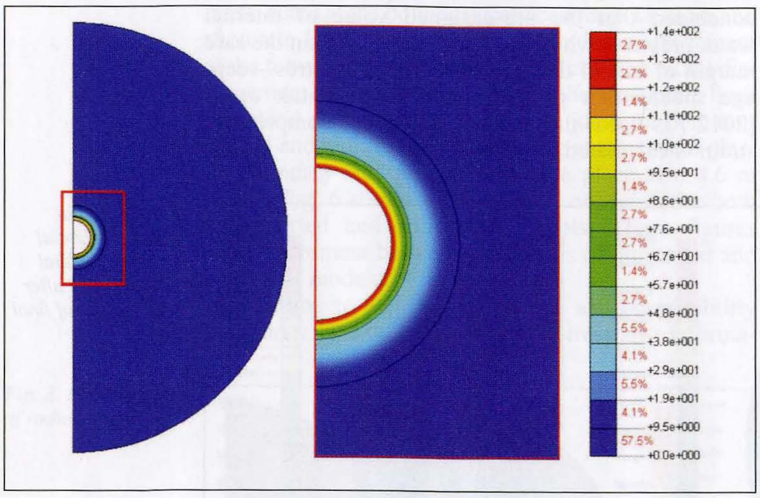

Fig. 10. Seepage pressure distribution.

9. Concluding remarks

A review of a two-dimensional plane strain FEM for the prediction of mechanical-hydraulic behaviour and bearing capacity of pre-stressed concrete-lined pressure tunnels has been reported. The distribution of stresses and deformations in the lining and rock mass as a result of tunnel excavation, installation of support and final lining, pre-stressing and the activation of internal water pressure is obtained by a phased analysis procedure. While the rock mass covering the pressure tunnel is modelled based on the non-linear yield function of Hoek-Brown rock plasticity, the shotcrete and final lining are modelled as elastic material and that obeys a combination of the yield criteria of Rankine and Von Mises.

The proposed modelling approach is capable of assessing the performance of pre-stressed concrete-lined pressure tunnels and overall, there is a global coherence between the results obtained using two-dimensional FEM and analytical solutions. For this reason, further studies involving FEM are encouraged to predict the behaviour of pre-stressed concrete-lined pressure tunnels in heterogeneous rock mass under various in-situ compressive stresses and different groundwater levels. ◊

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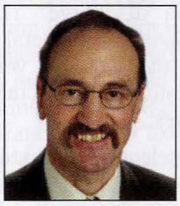
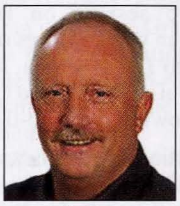


A.J. Schleiss



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