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An optimal method to design reinforced concrete lining of pressure tunnels

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Abstract
The utilization of the lining type in pressure tunnels is highly dependent on the geological and hydraulic conditions. There are two types of lining, namely concrete and steel lining but steel lining is one of the most expensive arrangements. To decrease the length of steel lining in these tunnels, the concrete lining, which prevents water seepage from the surrounding rock mass, is the appropriate alternative. In this work, a special attention is devoted to limit water losses in the concrete lining of pressure tunnel based on the critical reinforcing ratio in concrete lining. In order to evaluate the effect of internal water pressure on the permeability coefficient variation of the concrete lining and the surrounding rock mass, some simulations of reinforced concrete lining is implemented in the ABAQUS finite element software based on the coupled pore fluid-stress analysis. The results obtained indicate that although the critical reinforcing ratio has an important role in capturing the seepage flows and water losses, it is not sufficient to rely only on this parameter. However, among the various influential factors involved, a suitable arrangement of the reinforcement in the concrete lining should also be considered.

Keywords: *Pressure Tunnel, Reinforced Concrete Lining, Finite Element Model, ABAQUS Software, Seepage Control.*

1. Introduction

In a water pressure tunnel with concrete lining, part of the load will be transferred to the surrounding rock mass. If there is seepage flow through the lining, the excess pore water pressure will increase behind the lining [1]. Therefore, it is required to know the internal pressure in the tunnel design. Hence, it is also important to identify the permeability and deformation characteristics of the rock mass under the condition of high water head to estimate the seepage and stability of pressure tunnels. As mentioned above, the hydro-mechanical interaction analysis for the pressure tunnel and surrounding rock mass is important for two reasons. Firstly, the hydraulic characteristics of seepage phenomenon such as the efficiency of the concrete lining, rock mass, grouted zone, and seepage flow rate can be evaluated. Secondly, the effect of seepage force on the stress–deformation behavior of the concrete lining and the surrounding rock mass can be examined [2].

In the recent years, the concrete lining design of water pressure tunnels has been developed considerably. Seeber [3] has presented a diagram to evaluate the bearing capacity of pressure tunnel lining pre-stressed by grouting as long as in which the rock masses behave as an elastic material and the lining is impervious. Among the analytical methods, Schleiss [4-6] and Fernandez [7] have presented a framework to take into account the hydro-mechanical interaction in the reinforced concrete lining design of pressure tunnels, which seems to be a more wide-ranging solution. However, in those approaches, the elastic behavior of the reinforced concrete lining and the surrounding rock mass under internal water pressure are considered. Following the thick-walled cylindrical theory, Su and Li [8] have carried out a design approach of pressure tunnel with reinforcement concrete lining under consolidation grouting. Their results showed that the method of consolidation grouting could sharply decrease water seepage from the tunnel. In addition, the quality of consolidation grouting is more essential than the depth of grouting to control water leakage. Simanjuntak et al. [9] have developed the applicability of the finite element model to predict the hydro-mechanical behavior and bearing capacity of pre-stressed concrete-lined pressure tunnels subjected to a 2D plane strain condition. Parvathi et al. [10] have studied the effect of the size of the tunnel on the stress distribution in the concrete lining of pressure tunnels. Olumide [11] has simulated the coupling of stress, seepage, and lining crack propagation in the design of plain concrete lining of pressure tunnels using the Plaxis 2D elasto-plastic finite element program. The results obtained indicate that if the rock mass is originally tight or pre-stressed, the seepage from the tunnel will increase the external pressure. The external pressure decreases the gradient between the internal and external water pressure, which leads to a drop in the seepage flow of the tunnel. Simanjuntak et al. [12] have also determined the bearing capacity of concrete-lined pressure tunnels based on the superposition principle in non-uniform *in-situ* stress conditions. The results obtained demonstrate the different distributions of crack opening locations in the final lining, which is useful for taking measures regarding the tunnel stability. In order to simulate the boundary condition between the reinforced concrete lining and the surrounding rock and to compute the secondary permeability of cracked lining, Zhou et al. [13] have developed a water-filled joint (WFJ) element. They showed that the new equivalent coupling method reflected the actual stress level of reinforced concrete tunnel with a high internal water pressure.

In the present work, the variations in hoop stress with radial distance and shear stress along the concrete rock interface have been computed. In addition, the total amount of tension and compression stresses in the lining is applied in its design. Since the effect of discontinuities is assumed by changing the material behavior, the concrete lining and the surrounding rock mass are considered as equivalent continuum media, and a three-dimensional finite element method (3D FEM) is employed to determine the seepage flow. The proposed model also involves the effect of stress-dependent permeability of reinforced concrete lining and surrounding rock mass using a coupled pore fluid-stress analysis. It is worthy to note that in the previous models, it has not been considered. Based on the literature, the hydro-mechanical interaction of the previous works [6, 8, 11, 13] was based upon the indirect-coupled method. However, a change in internal water pressure incorporates changes in the volume of the media; therefore, the direct hydro-mechanical coupling should be taken in account. To conclude, this proposed method solves the equilibrium equations governing the pressure tunnel problems for the first time.

2. Effect of hydro-mechanical interaction on concrete cracking and seepage flow

The hydro-mechanical interaction is a complex process when the inner water is discharged through the cracked lining and the porous rock mass [14]. Before the lining cracking, the permeability coefficient of reinforced concrete lining is very small, and the pore water pressure in the lining is set to have a logarithmic distribution [4]. When the internal water pressure increases, the concrete lining will crack as soon as the tensile stress in the concrete lining exceeds its existing tensile strength [13]. After the crack develops and propagates, the external water pressure on the lining decreases the seepage flow from the tunnel. Finally, based on the continuity of flow, the water flow in the lining and the surrounding rock mass will tend to reach an equilibrium state [15].

2.1. Modification of permeability coefficient after media cracking

When cracks develop in the concrete lining at a high water pressure, the properties of the concrete lining change, and as a result, the stress dependent permeability of the lining and the surrounding rock mass in pressure tunnels should be considered in the computation [15].

The permeability coefficient controls the rate of seepage flow in the porous and fractured media. Although permeability represents an original property of the porous media, it can be modified when subjected to the stress variations [15]. Instead of changing aperture, the change in the void space or volume is the typical consequence that results in changing the permeability coefficient [16]. The permeability coefficient variation of the rock mass can be expressed as:

$$k = k_0[(\frac{1}{\phi_0})(1+\varepsilon_V)^3 - (\frac{1-\phi_0}{\phi_0})(1+\varepsilon_V)^{-(1/3)}]^3 \quad (1)$$

where k_0 is the initial permeability coefficient (m/s), ε_V is the volumetric strain due to the application of the internal water pressure corresponding to the evolution of plasticity, and ϕ_0 is the initial void ratio [17, 18]. The permeability coefficient variation of concrete lining is [19]:

$$k = k_0.\exp[11.3D]^{1.64} \quad (2)$$

where k and k_0 are the current and the initial material permeability, respectively, and D is the damage variable of the concrete lining. Since the response of the model is considered only in tension, the concrete damage extent is described by the following equation [20]:

$$D_t = D = 1 - \frac{\sigma_t}{E_0(\varepsilon_t - \varepsilon_t^{pl})} \quad (3)$$

where E_0 is the initial elastic stiffness of the concrete lining (MPa), σ_t is the tensile stress (MPa), ε_t^{pl} is the tensile equivalent plastic strain, and ε_t is the total strain.

3. Finite element model of pressure tunnel

The pressure tunnel of the Gotvand dam located in SW Iran is taken as a case study for the numerical simulation. The pressure tunnel is simulated using the ABAQUS finite element program. It is assumed that the tunnel has a circular shape with a diameter of 11 m, and is constructed at a depth of 110 m, as depicted in Figure 1. In order to simulate the infinite boundary conditions for the ground, a square block of 110 m in depth and width is selected for the ground, and the tunnel length is taken to be 1 m [21].

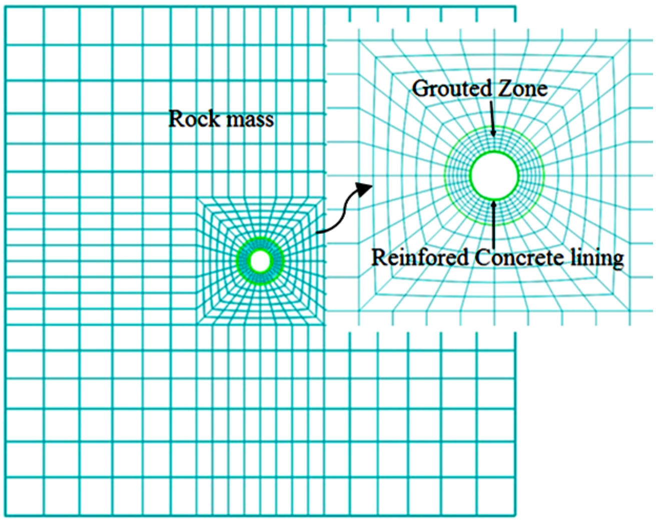


Figure 1. Finite element model.

Table 1. Mechanical parameters of material [21].

Parameter	Symbol	Unit	Rock mass	Concrete*	Reinforcement
Density	ρ	kg/m^3	2450	2500	7850
Poisson's ratio	ϑ	-	0.25	0.15	0.25
Young's modulus	E	GPa	3	20	200
Cohesion	C	MPa	0.01	4	-
Internal friction angle	ϕ	degree	35	40	-
Porosity	n	%	15	5	-
Permeability	k	m/s	2×10^{-6}	1×10^{-8}	-
Dilation angle	ψ	degree	6	25	-

***Concrete Damage Plasticity model parameters: eccentricity = 1, K (ratio of the second stress invariant on the tensile meridian) = 0.6667, f_{bo} / f_{co} (ratio of the initial equibiaxial compressive yield stress to the initial uniaxial compressive yield stress) = 1.16, viscosity parameter = 0.**

The rock mass is assumed to behave as elastic-perfectly plastic Mohr-Coulomb. In order to demonstrate the response of the reinforced concrete lining, the concrete damaged plasticity model along with the elasto-plastic behavior is considered to simulate the behavior of the concrete and reinforcement [15]. The evolution of the yield surface of the concrete lining is also controlled by tensile and compressive equivalent plastic strains, correspondingly [22]. The material properties of the rock mass, reinforcement, and concrete lining are given in Table 1.

Since the pressure tunnels have been excavated using the conventional drill and blast technique, in this work, the excavation steps are simulated by the stiffness reduction method. It should be noted that the reinforced concrete lining may be separated from the temporary support and/or the surrounding rock mass under the high internal pressure [15]. In order to simplify the hydro-mechanical interaction analysis, the concrete lining and the surrounding rock mass are assumed to be well-secured. In addition, the treatment of temporary support is considered as a part of concrete lining, and the thickness of the final lining is suggested to be 40 cm. The quality of contact surface between the reinforcement and concrete has a significant impact on the results of the analyses. If the contact area is reduced due to the large deformations, the effect of the discontinuity should be accounted in the analysis. The embedded element is used for making a full contact between the reinforcement and the concrete [15].

Based on the structural tunnel design and to control the compression and tension in the concrete lining [23], the reinforcement radius is taken at the two positions r_s = 5.15 m and 5.45 m, respectively. The 8-node trilinear displacement and pore pressure elements (C3D8P) are chosen for rock mass and concrete lining, while the 2-node linear truss elements (T3D2) are selected for reinforcement. In pressure tunnels, the internal water pressure at the inner surface of the lining is gradually applied to reach the maximum internal water pressure and steady state condition. For the phase of the internal water pressure loading, two boundaries are considered. The first boundary at the inner surface of the reinforced concrete lining is imposed by internal water pressure. The second boundary is imposed at the outside of the model domain by the groundwater level. Since the tunnel is assumed to be excavated under drained conditions, external water pressure at the outside of the model domain is equal to zero [15].

3.1. Verification of model

Since the lining and rock mass have non-linear properties and complex behaviors, for verification of the model in the ABAQUS software, the model is simulated with homogeneous, isotropic, and elastic behaviors. In Figure 2, the results of seepage flow on the interface of the concrete lining and rock mass obtained by the analytical and numerical solutions indicate that there is a ±6% difference between them. Then the results of the elastic behavior of the model show a good agreement with the results of the analytical solutions. Therefore, this numerical model has been employed for the non-linear analyses.

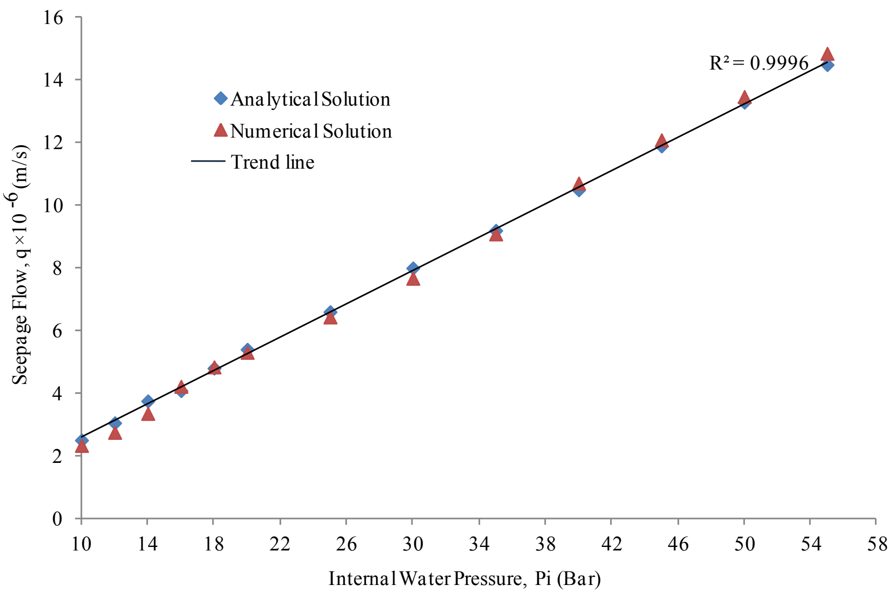


Figure 2. A comparison between the results of the analytical and numerical methods in the seepage flow calculation.

4. Results and discussion

4.1. Tunnel stability control before applying internal water pressure

In order to evaluate the pressure tunnel stability before applying the internal water pressure, it is required to compare the displacement results from the numerical model with the allowable displacements around the underground space [24]. To achieve the stability, it is necessary that the displacements obtained from the numerical model are less than the permissible displacement from Equation (4) [25]. This equation expresses the critical strain in the confined pressure conditions around the tunnel as a function of the zone deformation modulus [24].

$$\log \varepsilon_c = -0.25 \log E - 1.22 \qquad (4)$$

where E is the rock deformation modulus (kg/cm^2) and ε_c is the critical strain percent. By determining the permissible strain, the displacement will be calculated. The permissible displacement is obtained as follows [24]:

$$\varepsilon_c = \frac{u_c}{a} \qquad (5)$$

where u_c is the displacement on the tunnel roof and a is the tunnel radius (Sakurai 1997). Thus the critical strain is equal to 4.57 × 10^{-3} and the displacement has been calculated to be 25 mm. The displacement–recorded in the roof of the tunnel is investigated for the lining thicknesses of 30 cm, 35 cm, and 40 cm. As shown in Figure 3, the displacement in the roof of the tunnel is about 7 mm for the thickness of 40 cm. Thus the safety of the tunnel is not at risk before applying internal water pressure.

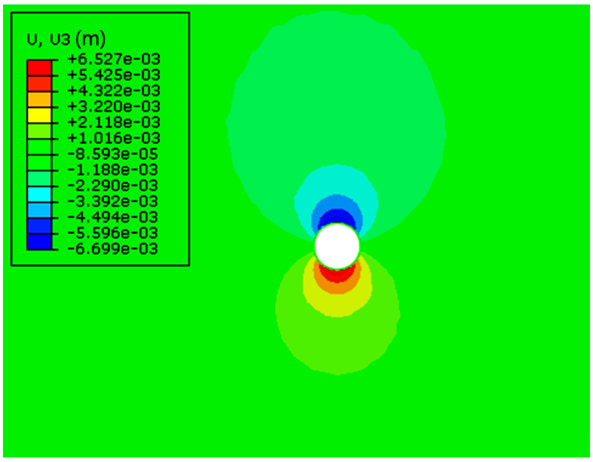


Figure 3. Displacement in the roof of tunnel before the applied internal water pressure.

4.2. Seepage control after applying internal water pressure

During the water filling process of the tunnel, when the tensile stress exceeds the tensile strength at the intrados of concrete lining, the cracks occur in the concrete lining under the hydro-mechanical interaction. The equivalent plastic strain in tension is referred to as the damaged state and the crack development in the concrete lining under the high internal water pressure. The development of the cracks due to the damaged states in the reinforced concrete lining is demonstrated in Figure 4. In these conditions, the stress in the steel bars increases in the locations of lining due to the internal water pressure, as shown in Figure 5.

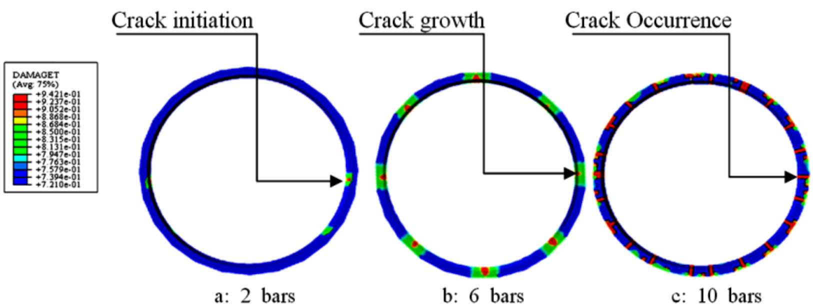


Figure 4. Development of cracks in the concrete lining under internal water pressure.

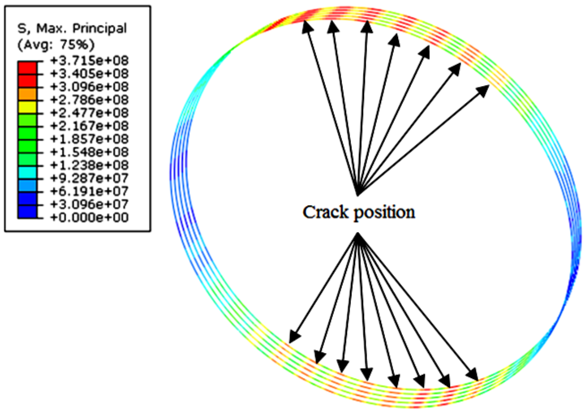


Figure 5. Increasing the stress on the steel bars under internal water pressure in the locations of lining.

4.2.1. Proposing reinforced concrete lining

Since the stresses in the suitable arrangement of reinforcement have to be lower than the allowable stress, and the allowable stress is smaller than the yield stress ($f_y = 400\text{MPa}$), there is no difference between the elastic and elastic-perfectly plastic behaviors of the reinforcement. Therefore, the stress-strain relationship of the reinforcement in the model is expressed as $\sigma = E\varepsilon$.

With respect to the thickness of the concrete lining, the maximum tensile stress in the concrete lining from the numerical method has been obtained to be about 1.46 MPa. The maximum tensile stress at the point of cracking is obtained from the following equation:

$$F_t = \sigma_t.A_c = 1.46\times10^6\times0.4\times1 = 584000(\text{N}) \qquad (6)$$

where F_t is the maximum force in the concrete lining (N), σ_t is the maximum critical tensile stress in the concrete lining (N/m^2), and A_c is the cross-section area of the concrete lining (m^2).

This maximum force in the concrete lining will be the criterion for the reinforcement design in the concrete lining. In the allowable stress design method, the stress induced at the steel bars should be less than the allowable stress ($f'_y = 400\text{MPa}$), and thus the percentage of the reinforcement in the concrete lining can be calculated using:

$$A_s = F_t / 0.5f'_y = 584000/(200\times10^6) = 0.00292(\text{m}^2) \qquad (7)$$

where A_s is the cross-section area of steel bars (m^2) and f'_y is the stress induced at the steel bars in the allowable stress design method (N/m^2).

The design of the reinforcement in the concrete lining of the pressure tunnels is governed by limiting stress in the steel bars, limiting width of cracks in the concrete lining, and limiting water losses from the tunnel [5]. To meet all the requirements, the spacing and the diameter of the steel bars in the concrete lining should be optimized for up to a particular level of internal water pressure.

Damage models of the concrete lining are capable of representing crack initiation and crack propagation within a continuum framework. In principle, these models do not provide crack openings, and the maximum crack width is not calculated. In the durability analyses of reinforcement concrete lining, however, the transfer properties are the key issues controlled by crack propagation and crack opening.

Stresses at different levels of internal water pressure for the cases of the same steel ratio are compared in Figure 6. As a result, the stress in the reinforcement of the Φ 16 mm @ 20 cm remains at a lower amount under 80 bars internal water pressure.

Based on Figure 7, the water losses from the tunnel for the two cases of steel quantities of Φ 16 mm @ 20 cm and Φ 20 mm @ 30 cm in the concrete lining are compared under 15 bars internal water pressure. Since the water losses of the tunnel for the two cases of steel bars are about 1.5×10^{-6} m^3/s and 2.1×10^{-6} m^3/s, respectively, the water loss from the tunnel with the net of Φ 20 mm @ 30 cm increases about 40%. Thus the distribution of the steel bars is not appropriate, and therefore, the net of Φ 16 mm @ 20 cm is the appropriate arrangement of reinforcement that contributes to minimize water losses from the tunnel.

The development of the cracks in the reinforced concrete lining under the internal water pressure of 1.5 MPa is given in Figure 8. The state of the Φ16 mm @ 20 cm contributes to distribute the cracks on the reinforced concrete lining in the form of micro-cracks. In this state, when the number of cracks increases, the cracks do not reach the lining. Therefore, the high internal water pressure on the reinforced concrete lining has a certain effect on the changes of permeability coefficient, and subsequently, seepage flow.

Figure 9 presents the combination of two outputs on a part of the concrete lining; it means that the tensile stress shows an increase on the steel bars in the locations of cracking of the concrete lining.

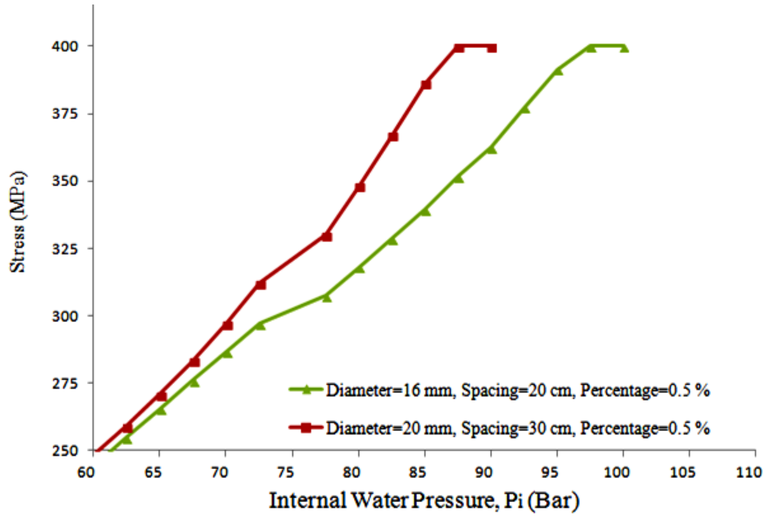
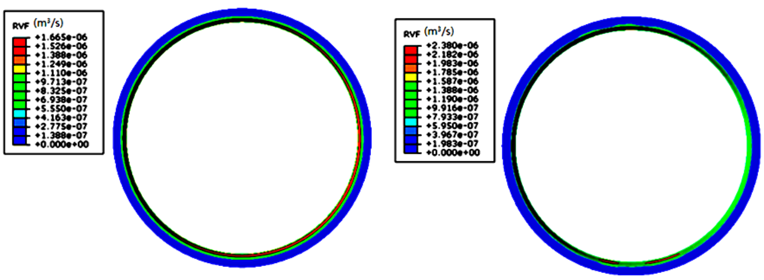
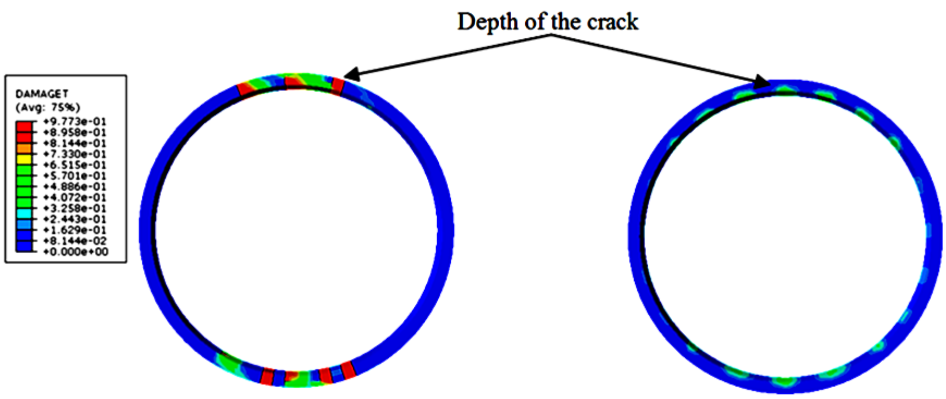


Figure 6. Comparison among stresses at different levels of high water pressure for the cases of the same steel ratio.



a: Φ16 mm @ 20 cm b: Φ20 mm @ 30 cm

Figure 7. Comparison between water losses of the tunnel.



a: Φ20 mm @ 30 cm b: Φ16 mm @ 20 cm

Figure 8. Comparison between the properties (depths and number) of cracks.

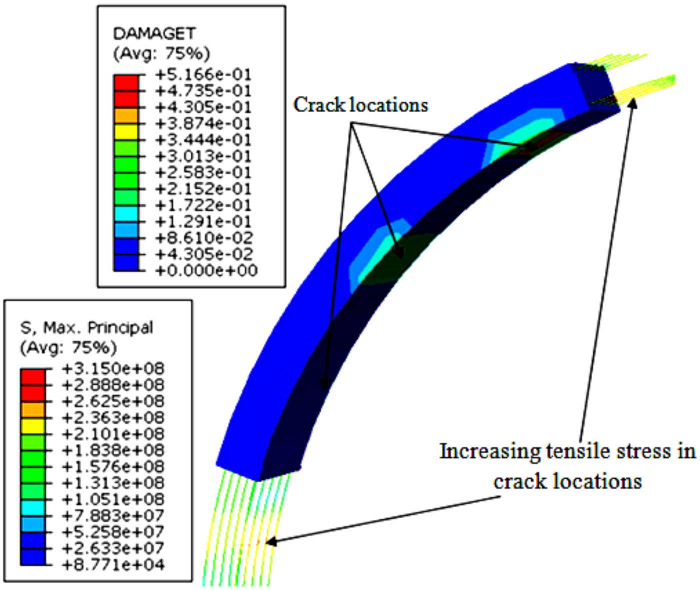


Figure 9. Increasing tensile stress at the location of cracking in concrete lining.

5. Conclusions

In the present work, the analyses were carried out for a pressure tunnel project as a part of the Gotvand HPP scheme, which is located in SW Iran. Pore fluid-stress analysis was carried out to propose the reinforced concrete lining of the pressure tunnels. In order to determine the more accurate response of the concrete lining and the surrounding rock mass under the internal water pressure, the strain-dependent permeability was employed based on the volumetric plastic strain in the plastic zone. Pore fluid-stress analysis was considered to compute the reinforced concrete lining of the pressure tunnels. According to the analysis performed within the continuum, adequate distribution of reinforcement and optimization of lining thickness were determined to control the lining crack. This could be done by reducing the equivalent damaged states and tensile stress limiting the concrete lining. Furthermore, in order to avoid any damage to the tunnel, the rate of water leakage must not exceed a critical level.

References

[1]. Wang, M.B. and Wang, G. (2013). A Modified Stress-Displacement Solution for a Pressure Tunnel with a Permeable Liner in an Elastic Porous Medium Based on a New Model. Rock Mechanics and Rock Engineering. 46 (2): 259-268.

[2]. Hosseinzadeh, A., Nobarinasab, M., Soroush, A. and Lotfi, V. (2011). Coupled Stress- Seepage Analysis of Karun III Concrete Arch Dam. Proceedings of the Institution of Civil Engineers-Geotechnical Engineering. 166 (5): 483-501.

[3]. Seeber, G. (1985). Power Conduits for High-Head Plants; Part One. Water Power & Dam Construction. 37 (6): 50-54.

[4]. Schleiss, A.J. (1986). Design of Pervious Pressure Tunnel. International Water Power and Dam Construction. 38: 21-28.

[5]. Schleiss, A.J. (1997a). Design of Concrete Linings of Pressure Tunnels and Shafts for External Water Pressure. Tunneling Asia, New Delhi, India. pp. 291-300.

[6]. Schleiss, A.J. (1997b). Design of Reinforced Concrete Lining for Pressure Tunnels and Shaft. The International Journal of Hydropower & Dams. 4: 88-94.

[7]. Fernandez, G. (1994). Behavior of Pressure Tunnels and Guidelines for Liner Design. Journal of Geotechnical Engineering. 120: 1768-1791.

[8]. Su, K. and Li, Y. (2012). Design of Pressure Tunnel with Reinforcement Concrete Lining under Consolidation Grouting. Advanced Materials Research. pp. 2731-2735.

[9]. Simanjuntak, T.D.Y.F., Marence, M., Schleiss, A.J. and Mynett, A.E. (2012). Design of Pressure Tunnels using a Finite Element Model. The International Journal of Hydropower & Dams. 5: 98-105.

[10]. Parvathi, I.S., Praveen, T.V. and Kumar, K.S. (2013). Effect of Rock Mass Quality and Tunnel Size on Lined Pressure Tunnels using FEM. Journal of Rock Mechanics and Tunneling Technology. 19: 1-17.

[11]. Olumide, B.A. (2013). Numerical Coupling of Stress and Seepage in the Design of Pressure Tunnel under to High Internal Water Pressure. International Journal of Engineering and Technology. 3: 235-244.

[12]. Simanjuntak, T.D.Y.F, Marence, M., Mynett, A.E. and Schleiss, A.J. (2014). Pressure Tunnels in Non-uniform In-situ Stress Conditions. Tunneling and Underground Space Technology. 42: 227-236.

[13]. Zhou, Y., Su, K. and Wu, H. (2015). Hydro-Mechanical Interaction Analysis of High Pressure Hydraulic Tunnel. Tunneling and Underground Space Technology. 47: 28-34.

[14]. Bian, K., Xiao, M. and Chen, J. (2009). Study on Coupled Seepage and Stress Fields in the Concrete Lining of the Underground Pipe with High Water Pressure. Tunneling and Underground Space Technology. 24: 287-295.

[15]. Dadashi, E., Noorzad, A., Shahriar, K. and Goshtasbi, K. (2017). Hydro-mechanical interaction analysis of reinforced concrete lining in pressure tunnels. Tunnelling and Underground Space Technology. 69: 125-132.

[16]. Bai, M., Meng, F., Elsworth, D., Zaman, M. and Roegiers, J. (1997). Numerical Modeling of Stress-Dependent Permeability. International Journal Rock Mechanics and Mining Science. 34: 2-10.

[17]. Jun, L.J., Bin, Y.X. and Zhou, Z.J. (2013). Numerical Simulation of Geo-stress and Pore Pressure Evolution around Oil or Water Well under Different Injection-Production Ratio. Mathematical Problems in Engineering. pp. 1-10.

[18]. Ying, C., Youzhi, W. and Yu, Z.Q. (2014). Coupled Seepage Elasto-Plastic Damage Analysis of Saturated Porous Media and its Application to Water Conveyance Tunnel. Tunneling and Underground Space Technology. 44: 80-87.

[19]. Cabot, P.G., Dufour, F. and Choinska, M. (2009). Permeability due to the Increase of Damage in Concrete: From Diffuse to Localized Damage Distributions. Journal of Engineering Mechanics. 135: 1022-1028.

[20]. Simulia. (2011). ABAQUS Theory Manual. Dassault Systèmes, Providence, RI, USA.

[21]. Mahab Ghods Consulting Engineering Company. (2008). Report of Introduction, Implemented Studies, and Execution Characteristics of Gotvand HPP Scheme. Tehran, Iran. pp. 6-38.

[22]. Ahmed, A. (2014). Modeling of a Reinforced Concrete Beam subjected to Impact Vibration using ABAQUS. International Journal of Civil and Structural Engineering. 4: 227-236.

[23]. ITA Working Group. (1988). Guidelines for the Design of Tunnels. Tunneling and Underground Space Technology. 3: 237-249.

[24]. Dadashi, E., Ahangari, K., Noorzad, A. and Arab, A. (2012). Support System Suggestion Based on Back Analysis Results Case Study: Babolak Water Conveyance Tunnel. Arabian Journal of Geosciences. 5: 1297-1306.

[25]. Sakurai, S. (1997). Lessons Learned From Field Measurements in Tunneling. Tunnelling and Underground Space Technology. 12 (4): 453-460.

ارائه روش بهینه‌ای برای طراحی پوشش بتنی مسلح در تونل‌های تحت‌فشار

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چکیده:

استفاده از نوع پوشش در تونل‌های فشار بسیار وابسته به شرایط زمین‌شناسی و هیدرولیکی است. دو نوع پوشش بتنی و فولادی وجود دارد؛ اما پوشش فولادی یکی از گران‌ترین طرح‌های پیشنهادی برای تونل‌های تحت فشار است. برای کاهش طول پوشش فولادی پیشنهاد شده در این تونل‌ها، پوشش‌های بتنی به عنوان مانعی برای نفوذ آب از توده‌های سنگی اطراف به داخل تونل گزینه مناسبی است. در این پژوهش، کاهش هدرروی آب از پوشش بتنی تونل تحت فشار بر اساس نسبت تسلیح بحرانی در پوشش بتن در دستور کار قرار گرفته است. لذا به منظور بررسی تأثیر فشار آب داخلی بر ضریب نفوذپذیری متغیر پوشش بتنی و توده سنگ اطراف، شبیه‌سازی پوشش بتنی مسلح در نرم‌افزار المان محدود ABAQUS بر اساس تحلیل توأمان تنش- فشارمنفذی اجرا شده است. نتایج به دست آمده نشان می‌دهد که هر چند نسبت تسلیح بحرانی پوشش نقش مهمی در کاهش جریان نشت و هدرروی آب از پوشش بتنی دارد، لذا تنها این پارامتر نتایج مطلوب را تأمین نمی‌کند؛ بنابراین لازم است تا در میان عوامل مختلف تأثیرگذار، آرایش مناسب تقویت‌کننده‌های پوشش بتنی نیز مورد توجه قرار گیرد.

کلمات کلیدی: تونل تحت‌فشار، پوشش بتنی مسلح، مدل المان محدود، نرم‌افزار ABAQUS، کنترل نشت.