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Numerical Coupling of Stress and Seepage in the Design of Pressure Tunnel under to High Internal Water Pressure

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ABSTRACT

This paper is based on the mechanism design of coupling stress and seepage in pressure tunnel, a suggested design criterion for pressure tunnel design procedure in stable rock conditions. Plain concrete lining of pressure tunnels are not absolutely tight and water seeps out of the tunnel resulting to loss of energy and often cause instability in the surrounding rock mass. Nevertheless, prestressing the surrounding rock mass by grouting keeps the seeped water within the vicinity of the tunnel and increase the external water pressure, thereby increasing the tunnel bearing capacity and reduce the seepage/water losses. The crack propagation and the influence of prestressing on the plain concrete lining have been studied and modeling of the phenomenon was performed by hydro-mechanical coupling of stress and seepage calculation performed using Finite Element program. The numerical coupling can be described as follows: the change of stress field changes of permeability coefficient and the change of the seepage field; the new seepage field tend to produce new seepage force and lead to the redistribution of stress field. The numerical model was used to replicate similar research work done and good agreement was recorded. Model was extended to practical example and finally matched with analytical solutions resulting in a rational methodology for the design of plain concrete lining under high internal water pressure of up to 35 bars. The results illustrate the applicability of the present method.

Keywords: *Pressure tunnel, numerical coupling, plain concrete lining, stress and seepage*

1. INTRODUCTION

The rapid development of water conservancy projects in the past years has necessitated the use of pressure tunnels with high hydraulic heads and large diameter in hydropower stations. The pressure tunnels are mostly lined by plain or reinforced concrete lining, but sometimes the tunnels can be left unlined or just lined by shotcrete. In extreme cases, where all other methods in term of lining strength or permeability cannot give satisfactory results, a tightening element is needed. The tightening element, thin or thick steel lining and in some cases plastic foil or plastic pipe is commonly used. Implementation of the tightening element increases the construction costs and minimizing of the tunnel length with tightening element is an important target by pressure tunnel design.

Lining has been widely used in pressure tunnel nowadays, since it can reduce the flow surface roughness and protect the surrounding rock from scoured by high velocity flow. The mechanism of inner water pressure acting on lining is based proposed theories that can be divided into two categories: the surface force theory and the body force theory. The surface force theory assumes that the lining is impermeable and the inner water load is treated as surface force is given by Zhang and Wu, (1980). This method is comparatively simple but not considering the influence of seepage field, especially when crack occur in concrete. The body force theory assumes that the lining is permeable and the inner water load should be treated as body force (Schleiss, 1986, 1987; Cao and Liu, 1991; and Ye, 1998, 2001). In this theory, the influence of seepage field and the hydraulic–mechanical interaction can be considered. Cracks often occur in the concrete lining under high internal water pressure, and therefore, the inner water flows out through the crack towards the rock mass. The material properties of the cracked concrete will change from approximately isotropic to anisotropic. As a result of this, the permeability characteristic and the constitutive model of the concrete should be reconstituted. For the whole system, the seepage field and the stress field affect each other until a new equilibrium between hydraulic and mechanical iterations reaches Kang et al., (2009); Busari and Marence, (2012). This process is very complex and many factors should be considered.

A number of fundamental criteria and other important considerations have to be defined during the pressure tunnel design. Marence, (2009) showed a possible flow chart defining design criteria that has to be taken into consideration during the power waterway design. The flow chart can be applied to each section along the power waterway and has to be included in the design of the vertical and horizontal tunnel alignment. The design approach that includes all important parameters to optimize the functionality of plain concrete lining in power waterway has been proposed by Busari and Marence, (2011).

Bearing of internal water pressure by plain concrete lined pressure tunnels is limited by the low tensile strength of concrete. Shrinkage of concrete and cooling of the lining by first filling causes a gap between the concrete linings and surrounding rock mass and therefore the surrounding rock mass cannot be included in the bearing of the internal pressure Seeber, (1985*a*, 1985b) and Busari and Marence, (2011). The low pressure grouting reconstitutes the contact with the surrounding rock mass and increases the bearing capacity, but still the bearing capacity of plain concrete lining is limited. The bearing capacity of the plain concrete lining can be considerably increased if the surrounding rock mass is radially grouted with high pressure grouting causing so called "pre-stressing" of the final concrete lining as shown by the analytical solution Seeber, (1985*a*) and numerically Busari and Marence, (2011, 2012). Such lining system dependent on the tunnel geometry and rock mass characteristics can be loaded by the internal pressures of up to 20 bars.

Nowadays, plain concrete lining are mostly pre-stressed by grouting. Grouting through radially set grout holes additionally increase the rock mass strength and stiffness, but also reduce the rock mass permeability. Reduced permeability of the rock mass gives possibility for additional effect that was up till now not used in the design of the lining. Relatively tight rock mass around the concrete lined tunnel reduces water losses and produces external water pressure that, as a contra-pressure, reduces the tensile stresses in the concrete lining. Including of the increased external water pressure (contra-pressure) caused by water seepage through the concrete lining in the design gives possibility to extend the applicability of the plain concrete lining and will allow estimation of the water losses through the concrete lining and could reduce the length of much more expensive steel lined sections. The simplicity of the coupling model assume that the concrete lining and the surrounding rock are well combined and the internal water pressure is jointly bear by both lining and surrounding rock make it more applicable to engineering analysis.

The numerical model is first used to replicate similar research work done and finally matched with analytical solutions resulting in a rational methodology for the design of plain concrete lining under high internal water pressure of up to 35 bars. In all the analyses the following assumptions have been made: (1) the rock mass behaviour is assumed to be in drained conditions; (2) lining material is elastic; (3) plane strain conditions apply at any cross-section of the tunnel; (4) deep tunnel, where the ground is considered weightless; the errors introduced are small for tunnels located at a depth of at least five times the tunnel radius (Bobet and Nam, (2007); (5) The stresses existing in the rock mass are related to the weight of the overlying strata and geological history. No geotechnical stresses are expected and the vertical stress is assumed as a weight of overburden. Ambient stresses are applied far from the tunnel and no displacement constraints at the boundaries.

2. BASIC REVIEW – ANALYTICAL METHODS

There are inevitably some small holes, joints or fissures in the surrounding rock mass, even if the lithology is good enough. Prestressing by consolidation grouting the surrounding rock mass is performed to improve the bearing capacity of final lining under high internal pressure which results into reduction in permeability and increase in stiffness and strength of the surrounding rock mass (Fernandez, 1994; Marence and Oberladstatter, 2005 and Busari and Marence, (2012).

2.1 Schleiss Theory

Theory is based on body force theory which assume that the lining is permeable and the inner water load should be treated as body force. A grout is required to fill on one hand the gap lining-rock and on the other hand fractures and large pores in the rock masses were discussed by Schleiss, (1986) and Kocbay et al, (2009). The total with crack in lining is a function tangential displacement of the rock mass and it is estimated by:

$$(2a) = u(r_a)2\pi/N \tag{1}$$

The number of cracks is governed by the weak zones in the lining and cracks are mostly found at crown of tunnel and transition invert in a plain concrete lining was highlighted Schleiss, (1997).

If the crack width is known, and the assumption of lamina and parallel flow can be applied in the crack. The water losses through the liner can be calculated by equation (2):

$$q = \frac{(p_i - p_a)(2a)^3\, n}{12\nu\rho_w(r_a - r_i)} \tag{2}$$

The water loss through rock mass in a tunnel above groundwater level is obtained from equation (3):

$$\frac{p_a}{\rho_w g} - \left(\frac{3}{4}r_a\right) = \frac{q}{2\pi K_r} ln \frac{q}{\pi K_r\, r_a} \tag{3}$$

$where \quad p_a = f(2a)$ is the water pressure on the outer side of the liner.

$$p_a = \frac{p_i}{1 + \left(K_r ln\left(^{r_a}/_{r_i}\right)/\left(K_c ln\left(^{R}/_{r_a}\right)\right)\right)} \tag{4}$$

$where\ K_r\ and\ K_c$ are permeability of rock and concrete liner respectively;

$r_i, r_a\ and\ R$ are the internal radius of the lining, external radius of the lining and external radius of the rock zone affected by seepage respectively. $R = 2r_a$ can be assumed for pervious rock ($K_r \geq 100K_c$). For tight rock ($K_r \leq K_c$), $R = 10r_a$ give good result see Schleiss, (1986).

Based on the thick-walled cylinder theory, radial deformation of the rock zone influenced by seepage, $u(r_a)$, is calculated as follows:

$$u(r_a) = [p_a - (\rho_w.g.b)]C_1 - p_r(R)C_2 - [p_r(R) - \sigma_r(r_a)]C_3 \qquad (5)$$

The value of R for tunnel above groundwater level can be obtained as follow Schleiss (1997):

$$R = {}^{q}/_{\pi K_r} \ln 2 \qquad (6)$$

Besides p_a, the mechanical boundary pressures at the inner and outer surface of the rock zone are influenced by seepage, $\sigma_r(r_a)$ and $p_r(R)$, have to be considered in equation 4. The radial stress transmitted to the rock mass by cracked concrete is calculated from equation 7:

$$\sigma_r(r_a) = p_r(r_a) = \frac{1}{2}(p_i - p_a)\left[1 + \frac{r_i}{r_a}\right] \qquad (7)$$

The boundary pressure, $p_r(R)$ between rock zone affected by seepage and rock zone not affected is obtained as:

$$p_r(R) = [p_a - (\rho_w.g.b)]C_4 - [\sigma_r(r_a)]C_5 \qquad (8)$$

Where C_4 and C_5 are computed from equation 9 and 10 respectively:

$$C_4 = \frac{1}{2(1-\upsilon_r)}\left[\left(\frac{r_a}{R}\right)^2 + \frac{(R^2 - r_a^2)(1-\upsilon_r)}{2R^2 ln\left(\frac{R}{r_a}\right)}\right] \qquad (9)$$

$$C_5 = \left(\frac{r_a}{R}\right)^2 \qquad (10)$$

Water losses through concrete liner, grouted zone and rock mass zone are computed iteratively from equation 11.

$$\frac{p_i}{\rho_w g} - \left(\frac{3}{4}r_g\right) = \frac{q}{2\pi K_r} ln\frac{q}{\pi K_r r_g} + \frac{q}{2\pi}\left[\frac{1}{K_c} ln\left(\frac{r_a}{r_i}\right) + \frac{1}{K_g} ln\left(\frac{r_g}{r_a}\right)\right] \qquad (11)$$

$where\ r_g\ and\ K_g$ are the radius and permeability of the grouted zone respectively.

According to Schleiss, (1987), the crack of the grouted zone can be prevented by injecting pressure as high as the tensile stress generated by the internal water pressure at the outer surface of the liner.

3. NUMERICAL METHODS

3.1 Model Set-up

With finite Element Plaxis 2D computer program, a model was set up and tested. The model was the use to replicate numerical analyses from previous work under the same experimental conditions to verify the performance of the program. The replicated works includes i) the maximum load computation in a shallow tunnel by Lee and Nam, (2006) ii) Seepage forces acting on the lining by Lee and Nam, (2006). Furthermore, the seepage losses obtained from numerical analysis is compared with the calculated one using analytical solution (equation 11). Finally, simulation of permeable plain concrete operational loading of high internal water pressure was executed using existing pressure tunnel material parameter. A schematized conceptual model set up methodology is presented in Figure 1.

Distributed load model (Figure 2) was set up and compared with Full mesh generating model. The latter present modeling of real state of rock mass in terms of height (for example deep tunnels) while in the former, part of model height is reduced to cater for the situations where model grid becomes too dense to display and can save computation time without jeopardizing the accuracy of the results. The model simulates deep excavation thereby overcoming one of the shortcomings of PLAXIS 2D - shallow tunnel. The full mesh generating model area covers 250m height and 100m in width. For this research, a distributed load mesh generating model has been used. The tunnel tube is circular with internal radius of 3.0m and the mesh is seven (7) times more than the tunnel diameter in all directions – the deformations outside the specified area can be neglected and appropriate boundary conditions have been used.

The material and loading modeling has been discussed Busari and Marence, (2011). Therefore, much emphasis will be placed on the simulation of internal water pressure.

PLAXIS 2D is designed for shallow tunnel but can be use for model up to 200m height as found in the full model approach of this research work. When the model becomes too high (say above 200m) and the grid is too dense to display, the upper part of the model can be omitted. The weight of the rock mass that makes the upper part must be compensated for to avoid the generation of unrealistic stresses.

Since the pressure in the rock is proportional to the depth of the overlying strata. A thin layer of thickness says $h_{virtual} = 1$ is created on top of the model to cater for the omitted part. The soil weight in the thin layer is modified as $\gamma_{virtual}$ and is given by:

$$\gamma_{virtual} = {}^{\gamma_{real} \times h_{real}}/_{h_{virtual}}$$
$$= 26 \times {}^{100}/_{1} = 2600kN/m^3$$

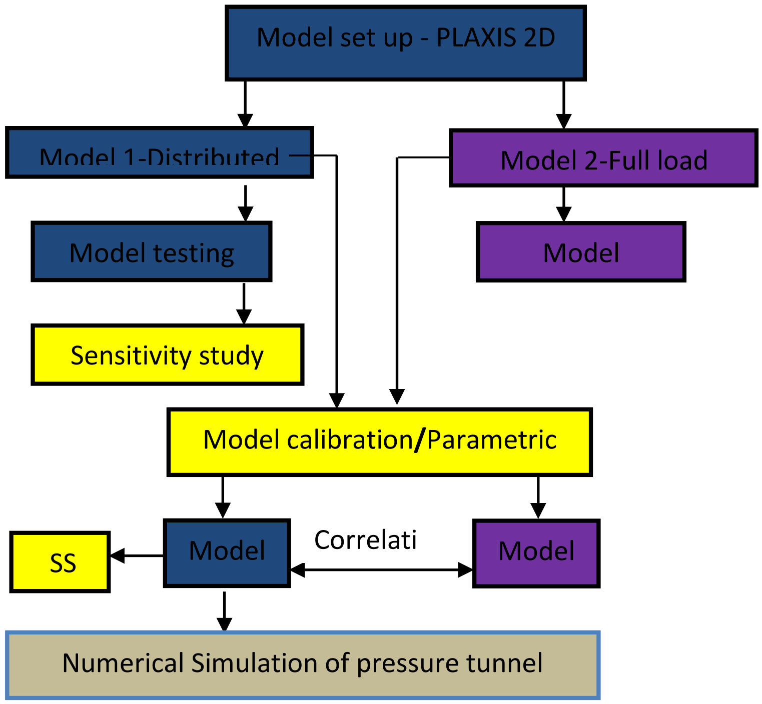

Figure 1: Conceptual model set up

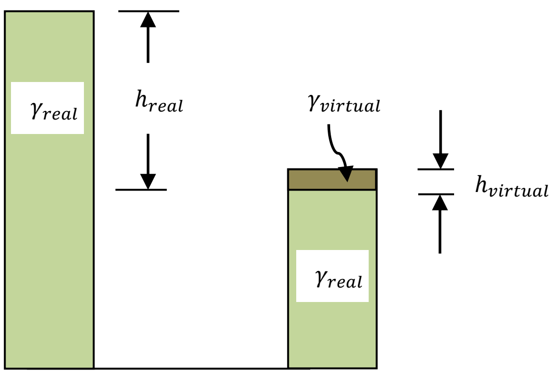

Figure 2: Theory of deep tunnel simulation

(Distributed Load Model)

The tunnel is excavated by the tunnel boring machine (TBM) with the following main geometric data: Overburden height (h) = 200m; Internal tunnel radius (r) = 3.00m, groundwater level is below the tunnel. The mesh consists of approximately 4000, 15-nodes as the basic element type. The global mesh is set to fine and, clusters and lines refined. The materials are mainly rock and concrete (see Table 1). The meshing model and boundary condition are as shown in figure 3.

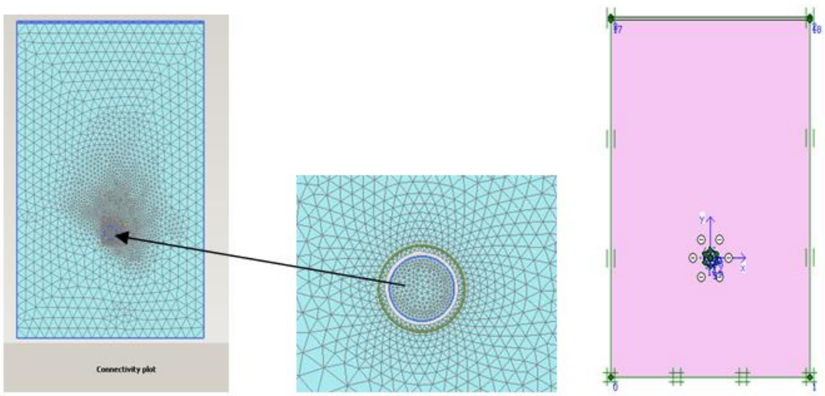

Figure 3: Meshing of Finite Element model and boundary conditions

Table 1: Material parameters

Parameters	*Symbol*	*Rock mass*	*Shotcrete*	*Final lining*	*Unit*

Modulus of elasticity	E	10	20	30	GPa
Poisson's ratio	υ	0.20	0.22	0.22	-
Unit weight	γ	26	24	24	kN/m^3
Frictional angle	ϕ	40	40	40	°
Cohesion	C	1000	1000	1500	kN/m^2
Thickness of lining	d	-	0.1	0.3	m
Weight	w	-	2.4	7.2	$kN/m/m$
Thermal coefficient	α		1.2×10^{-5}		$/°C$

(Source: Ermenek Pressure Tunnel Project, Turkey, 2003)

3.2 Boundary Conditions

Horizontal displacement is prevented along the vertical edges of the mesh boundary (horizontal fixity $u_x = 0$).The vertical edges of the whole area were fixed against horizontal displacement and bottom end was secured against vertical displacement (Vertical fixity $u_y = 0$).Standard fixity of boundary edges.

3.3 Loading Steps

The following computational phases have been performed:

Loading 1: primary state of stresses (K_o =0.8)

Loading 2: initial stress relief (0.6)

Loading 3: simulation of excavation

Loading 4: shotcrete lining simulation

Loading 5: Final lining simulation

Loading 6: Grout modeling

Loading 7: Internal water pressure.

4. RESULTS AND DISCUSSIONS

4.1 Performance Results of Full Model and Distributed Load Model

The result of significance of load reduction factor on relationship between axial forces in the shotcrete and total deformation for the models is presented in Figure 4. The result of distributed load model using the same material properties provide almost the same result with the calibrated result of full model (see Busari and Marence, 2011) under the same load reduction factor. The load reduction factor, $\beta = 0.6$ gives the almost the same values for the inner force and total deformation at the secondary equilibrium state. The result showed a perfect correlation between the two models. Hence, both models are found adequate for further analysis.

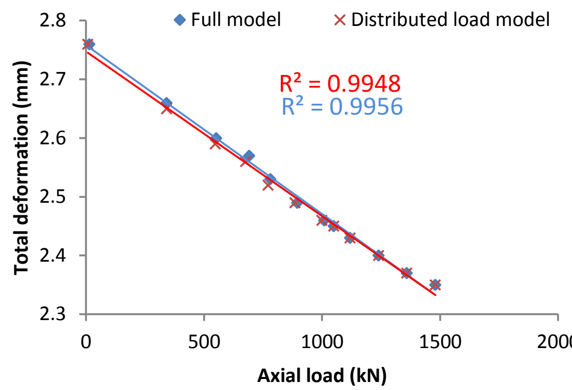

Figure 4: correlation between internal force and deformation

4.2 Maximum Load Computation in a Shallow Tunnel

First and foremost, the research work of Lee and Nam, (2006) is replicated under the same ground condition, model assumption and elasto-plastic Mohr Coulomb model. A scenario of circular drainage-type tunnel under the groundwater level was taken to examine the effect of seepage forces on the tunnel lining. To similate nature, the numerical analyses were performed three (3) drainage cases of a tunnel namely: dry condition; drainage with consideration of for the seepage force and waterproof (WP) concept with consideration for the hydrostatic water pressure. The case 1, was simulated by activating dry condition in the tunnel cavity and around the soil mass. Case 2, was simulated by deactivating the interface element in the lining thereby allowing seepage into the cavity. Case 3 was simulated by activating the interface element thereby making the lining impermeable. The analysis condition is shown in Figure 5.

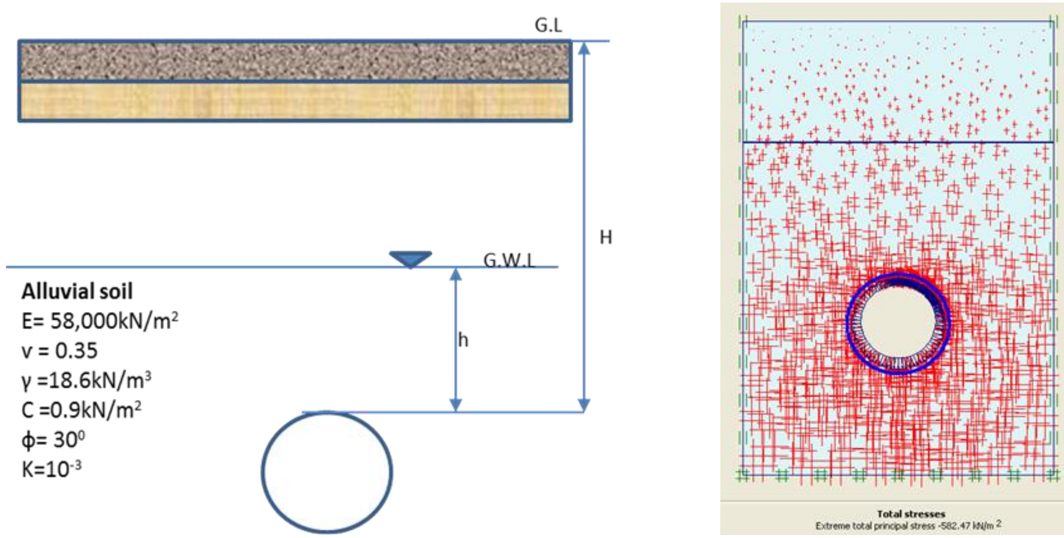

Figure 5: Analysis condition and FE Model

Plaxis 2D finite Element Program was used to simulate the scenario because it is possible to perform seepage analysis as well as hydromechanical analysis. The two analyses were super-imposed to obtain the seepage forces. The non-linear elasto-plastic Mohr –Coulomb model was used to for the calculation of stress change during excavation. The load distribution is shown in figure 6. The steady state equation was solved and pore water pressure stored at all nodes.

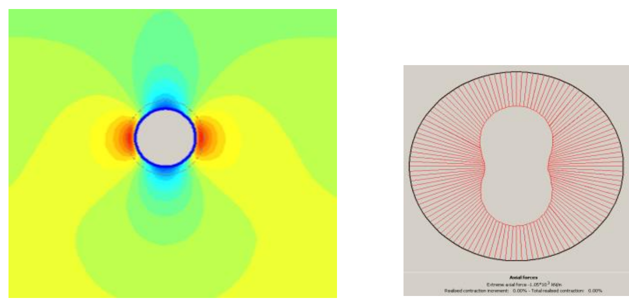

Figure 6: Load distribution on the lining (a) Stress redistribution (b) Axial load envelope

The lining is considered water proof, that is the tunnel is designed to support the hydrostatic water pressure. The difference in loads acting on the lining is presented in Table 2.

Table 2: Maximum loads acting on the tunnel shotcrete lining (WP)

Tunnel depth (H/D)	Ground water level (h/H)	Axial force (kN)		Stress kN/m²	
		Lee & Nam, (2006)	Busari	Lee & Nam, (2006)	Busari
2.0	0.5	97	100	350	370
2.0	1.0	156	160	566	590
2.0	1.5	216	220	764	786
3.0	0.5	127	128	460	470
3.0	1.0	216	220	786	791
3.0	1.5	305	312	1080	1095
4.0	0.5	156	160	571	586
4.0	1.0	276	280	1010	1018
4.0	1.5	395	402	1402	1411

The stresses of the tunnel lining were calculated according to the three drainage cases with variation in the groundwater level (h) and tunnel depth (H). The groundwater flows into the tunnel drainage system and the seepage forces are generated due to the difference of total head from the surrounding ground to the tunnel lining. The Plaxis results show a good agreement with results of Lee and Nam. For the waterproof case Table 2, the results showed that tunnel lining must support the hydrostatic water pressure and the axial stresses were more dominant than the bending. The reverse is the case for drainage case (see Lee and Nam, 2006). A maximum of about 2% difference in axial force was measured from the two results when compared and about 6% difference in stresses was recorded. The result showed a good correlation between this study and the report by Lee and Nam, (2006).

4.3 Seepage Forces Acting on the Tunnel Face

The second verification of Plaxis 2D finite Element program is the seepage force calculation. The properties of the ground material used for analysis are presented in Table 3.

Table 3: Properties of the ground

Soil type	Unit weight (kN/m^3)	Cohesion (kN/m^2)	Friction angle
Sand	15.2	0.0	35^0

The geometry, flow and boundary conditions for modeling the seepage acting on the tunnel face with variation of the groundwater height (h) and tunnel depth (H) under steady state condition is given in Figure 7.

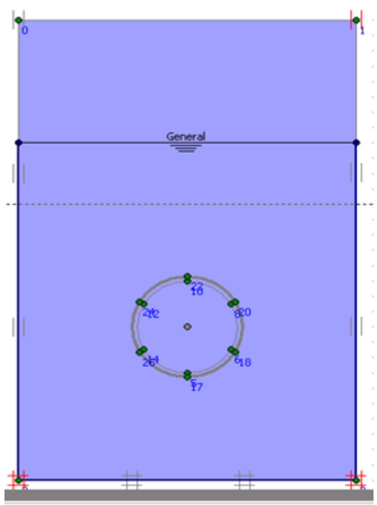

Figure 7: Geometry, flow and boundary conditions

In this study, two types of tunnels were considered as presented by Lee and Nam, (2006) – a drainage type tunnel and a water-proof type tunnel. In the case of drainage type, it is assumed that groundwater flows into all of the excavated surfaces including the tunnel face. The values of average seepage pressures during excavation calculated from numerical analyses are presented in Figure 8. It can be seen from the figure that the average seepage pressures have an almost linear relation with h/D ratio for both cases, although, the water-proof type has higher values than the drainage type. The results from Plaxis perfectly match with the findings of Lee and Nam, (2006). The ranges of the average seepage pressure were between 18 and 48 $\mathrm{kN/m^2}$ for the drainage type tunnel and, between 22 and 63 $\mathrm{kN/m^2}$ for the case of water-proof tunnel. The results were perfectly matched.

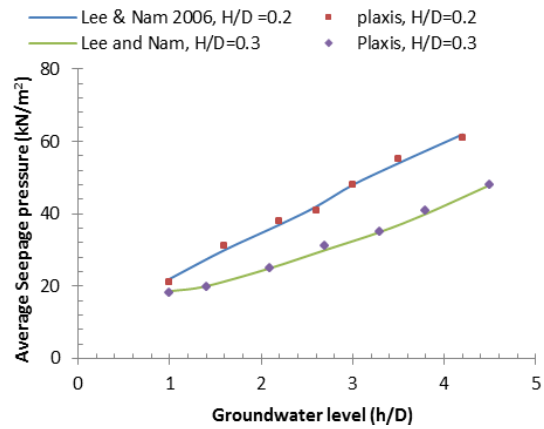

Figure 8: Seepage pressure with normalized Ground water level

4.4 Application to Deep Tunnel under High Internal Water Pressure

Numerical design of deep tunnels consists of simulation of the construction and operational loading. In this study, the rock mass behaviour is approximated using non-linear Mohr-Coulomb model. Model parameters (for the rock mass and support measures) are obtained from Ermenek tunnel project, Turkey. The material parameters are as presented in Table 1.

The operational loading of internal water pressure (IWP) is the most significant loading condition for the final lining. The internal pressure produces tensile stresses in the lining. The simulation of water losses or (seepage flow) is done using flow mode in water flow analysis. In addition, the stresses in the lining and rock mass due to internal water pressure are simulated using the consolidation analysis calculation type. The lining is 30cm thick and not primary lining was installed.

A grout in form of positive volumetric strain $\varepsilon_v = 0.2\%$ (approximately 15 bars of injection pressure) which is equivalent to $0.280m^3/m$ volume change in the rock mass was imposed in the rock cluster of twice the tunnel radius to simulate the prestressing effect - mechanical processes of reducing the permeability of the surrounding mass during loading operation.

Table 5: Stresses (kN/m^2) in the lining

S/N	K_c/K_r	*Lining* - Crown (Element 1137)	*Lining* (Right side) (Element 893)	*Lining* (Left side) (Element 1089)
1	1.000	3162	3715	4335
2	0.100	6474	6800	7015
3	0.020	8329	8828	8678
4	0.002	11821	12210	12022

4.5 Consolidation and Seepage Flow Analyses

Simulation of stresses and seepage flow in lining due to high internal water pressure

$$p_{i1} < p_{i2} < p_{i3} < p_{i4} < p_{i5}$$

and

$$K_{c1} = K_{c2} < K_{c3} < K_{c4} < K_{c5}$$

When the permeability coefficient of the liner is much smaller than the surrounding rock, more proportion of pressure induced by high inner water level will be bear by the lining, hence, the concrete lining can be assumed impermeable. On the other hand, when the permeability coefficient of the concrete is close to the surrounding rock, the lining can be treated as permeable member, and consequently more proportion of inner water pressure will be bear by the surrounding rock. In the comparison, the ratio of the permeability coefficient between the concrete and surrounding rock K_c/K_rranges from 0.02 to 1, and the permeability coefficient of the surrounding rock is constant 0.0000015 m/s. The stress variation in lining with different ratio of permeability coefficient is shown in Table 5.

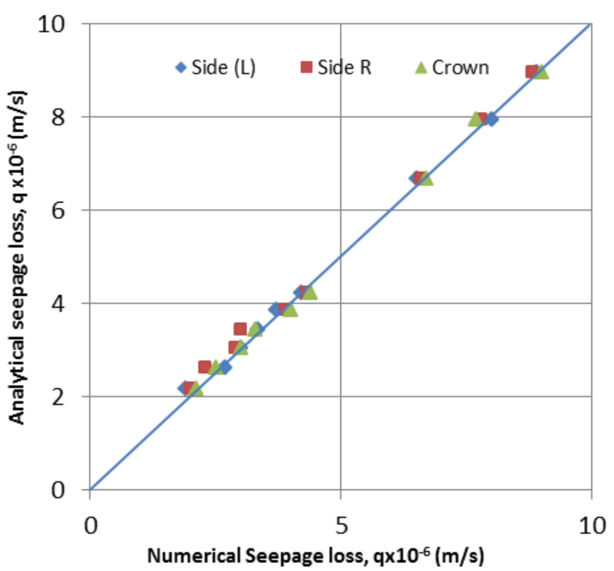

Figure 9: Comparison between the numerically computed seepage losses and those calculate by Schleiss analytical method

In Figure 9, the numerical value of seepage flow and the calculated ones by equation 11 are plotted. The results showed a good correlation with the line of perfect agreement. The analysis, therefore suggested that numerical computed values can be used as a measure of water losses in plain concrete lining. The water losses are computed and the result is presented in Table 6.

Table 6: Water loss through lining

i	Pressure (bars)	Numerical	Analytical
		water loss, q_w *l/s/km/bar*	
1	10	0.91	1.230
2	12	0.92	1.230
3	14	0.93	1.230
4	16	0.94	1.220
5	18	0.94	1.210
6	20	0.95	1.193
7	25	1.40	1.511
8	30	1.42	1.499
9	35	1.50	1.449

$$Water\ losses\ = q \times \pi D d \times 10^6/pi\ \ l/s/km/bar$$

Where D= 6m, tunnel internal diameter and d = 30 cm, equivalent thickness of the lining

From Table 5, the stress transformation in the lining due to imposed outward pressure of inner water force changes the flow matrix and water leaked out of the tunnel. As can be seen from Figure 10, the leaked water stayed in the vicinity of the tunnel as steady state is reached. The leak – out water are confined within radial zone of the grout.

From Table 6, even though with cracks (Figure 11), the seepage is flow is minimal and the stability of the rock mass is not disturbed and the lost water is found to be within the acceptable range 1-2 *l/s/km/bar* in practical criteria of technically tight lined tunnels (table 6).

5. CONCLUSION

The subjection plain or reinforced concrete lining to high water pressure always result to crack in the concrete lining. Consequently, the inner water leaks through the cracked lining towards the rock mass. If the rock mass is originally tight or prestressed, the leak out water stays in the vicinity of prestressed rock mass and increased the external pressure. The increased in external pressure decreased the gradient between the internal and external water pressure. Hence, the seepage loss is reduced. The hydraulic-mechanical interaction involved has been simulated using Plaxis 2D elasto-plastic finite element program. The coupling of the stress and seepage field is done to simulate the lining crack process of plain concrete pressure tunnel. The validity of the program has been checked by solving previous research problems and results stress and seepage forces compared with two calculation conditions. The model results for seepage analysis was further compare with analytical method and the results showed a very strong correlation. Both results from analytical and numerical methods were found to be in the acceptable range for technically tight lined tunnel.

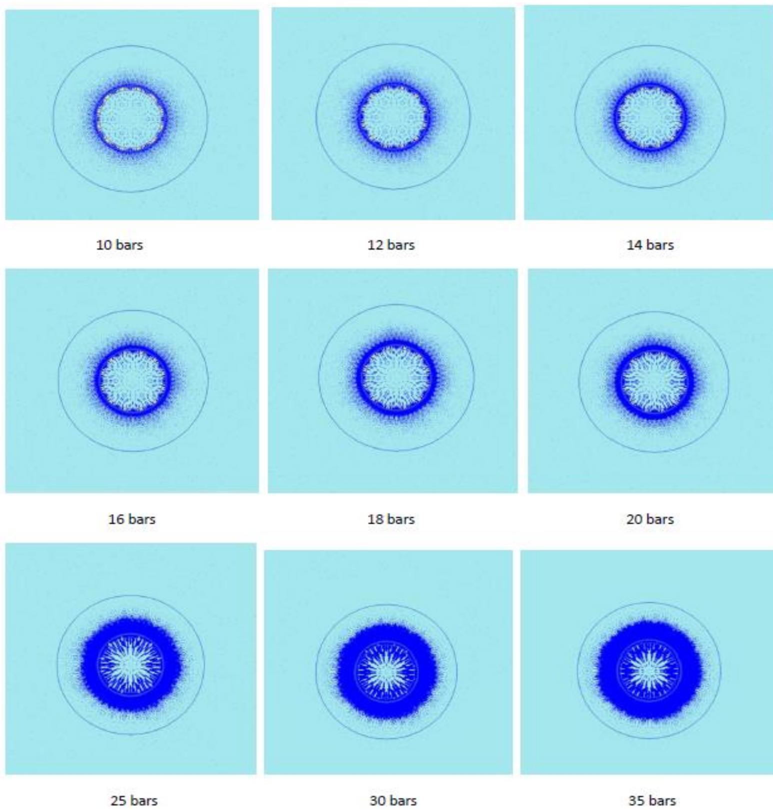

Figure 10: Seepage flow pattern in tunnels under high inner pressure with prestressed rock mass

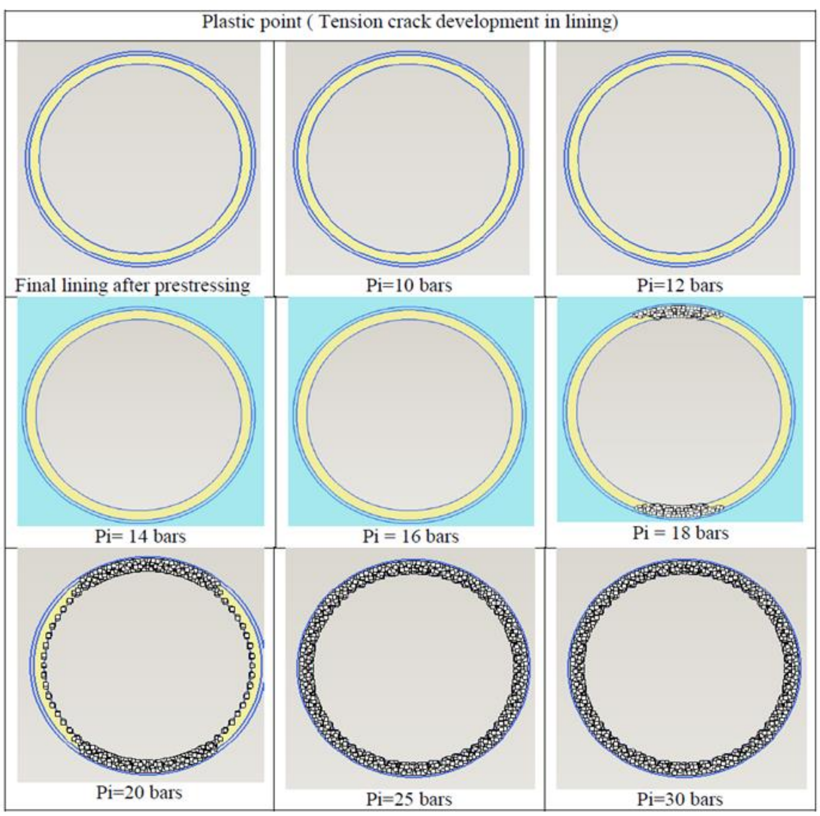

Figure 11: Cracks in plain concrete tunnels under high inner pressure

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