Mechanical-hydraulic interaction in the lining cracking process of pressure tunnels

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Pressure tunnels are in operation subjected to internal water pressure. When the hoop tensile stress acting at the lining intrados exceeds the tensile strength of concrete, longitudinal cracks occur in the concrete lining. As a consequence of crack openings, the internal water pressure will act at the lining extrados and cause high local water losses. If left untreated, these losses will induce the washing out of joint fillings and increase the risk of hydro-jacking of the surrounding rock mass. When pressure tunnels are situated close to valley slopes, excessive water losses can endanger the stability of the rock mass and provoke landslide.

Whether or not the internal water pressure is fully effective at the lining extrados, it depends predominantly on the number of cracks and the width of crack openings. The width of cracks can be estimated based on the total circumferential deformation of the rock mass, which is governed not only by mechanical boundary pressures, but also by seepage pressures. In turn, seepage pressures generate water losses from the tunnel, which are depending not only on the permeability of the rock mass, the grouted zone, and the concrete lining, but also, if any, on the width of the crack openings. The estimation of pressures transmitted to the rock mass requires therefore solutions using iterative methods dealing with this coupling behaviour.

This paper presents a method to estimate the distribution of seepage pressures and water losses around concrete-lined pressure tunnels pre-stressed by grouting, which considers the lining cracking process due to a high internal water pressure. The mechanical and hydraulic behaviour of cracked concrete-lined pressure tunnels is presented so as a first step for more elaborate numerical studies.

1. Basic Principle and Assumptions

A proper estimation of seepage pressure distributions throughout concrete-lined pressure tunnels pre-stressed by grouting is of great importance in investigating water losses and the overall stability of the tunnel against hydro-jacking. As long as longitudinal cracks in the lining can be avoided, the methodology proposed in Simanjuntak et al. (2012) to predict the bearing capacity of the so-called passive pre-stressed concrete-lined pressure tunnels is adequate. If longitudinal cracks occur in the lining, the prediction of water losses built up around these tunnels requires calculation methods, which involve not only seepage pressures acting throughout permeable materials but also seepage pressures associated by longitudinal cracks.

This paper presents an overview of design guidelines for passive pre-stressed concrete-lined pressure tunnels situated above the groundwater level. Regarding the bearing capacity of these tunnels, the concept is oriented towards the maximum utilization of the tensile strength of concrete. There are three zones considered in the analysis, namely the concrete lining, the grouted zone and the rock mass. While the compatibility condition is applied to reveal the unknown mechanical boundary pressures among these zones, the continuity condition is used so as to quantify the unknown seepage pressures through them.

Regarding the estimation of pre-stress-induced hoop strains in the lining, one can employ the load-line diagram method (Fig. 1b) suggested by Seeber (1985a). Seepage pressures around the tunnel and therefore seepage-induced hoop strains in the lining can be calculated using formulae given in Schleiss (1986b) assuming the radial flow out of the tunnel. In this paper, instead of using an arbitrary estimate of the reach of seepage flow, seepage-induced hoop strains are determined based on the magnitude of water losses and incorporated in the calculation of seepage pressures acting in the rock mass.

The maximum bearing capacity of passive pre-stressed concrete-lined pressure tunnels is derived based on the superposition principle, that is, the sum of pre-stress- and seepage-induced strains at the lining intrados. Both of these parameters are explained in more detail in the following sections.

2. Pre-stress Grouting

If the rock mass quality is favourable, a concrete lining can be pre-stressed by injecting the circumferential gap between the lining and the rock with a high pressure cement grout (Fig.1a). The value of the grouting pressure can be so high so that the compressive stresses because of pre-stressing do not exceed 85 per cent of the cube strength (Matt et al., 1978). Based on the thick-walled cylinder theory, the pre-stress-induced hoop strain at the lining intrados, $\varepsilon^{i}_{c,pinj}$, can be calculated as (Seeber, 1985a; Seeber, 1985b; Seeber, 1999):

$$\varepsilon^{i}_{c,\,p_{inj}} = \frac{p_{inj}}{E_c}\left(\frac{2\,r_a^{\,2}}{r_a^{\,2} - r_i^{\,2}}\right) \tag{1}$$

The pre-stress-induced hoop strain at the lining extrados $\varepsilon^{a}_{c,pinj}$, can be obtained by multiplying Eq. (1) with the factor $[(r_a^2 + r_i^2) - \nu_c\,(r_a^2 - r_i^2)]/2r_a^2$.

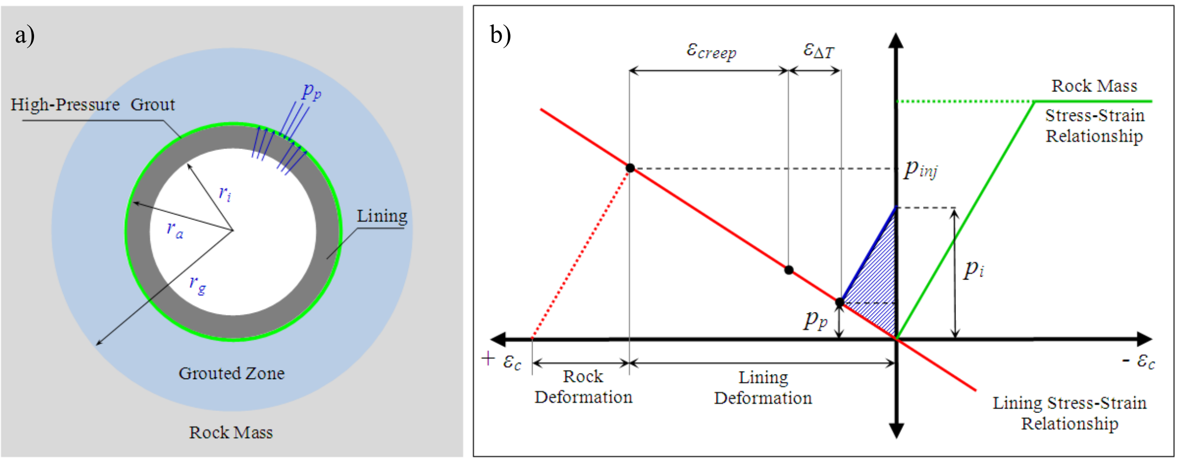

Fig. 1. (a) Pre-stress Grouting, and (b) Load-Line Diagram (after Seeber (1985b))

The strain loss as a result of temperature change at watering-up can be derived as a product of the temperature change, ΔT, and the thermal expansion coefficient for concrete, α_T. In the rock mass, approximately one-third of these losses can be expected. The total strain losses due to the thermal cooling, $\varepsilon_{\Delta T}$, can be expressed as (Seeber, 1999):

$$\varepsilon_{\Delta T} = \alpha_T\left(\Delta T + \frac{\Delta T}{3}\right) \tag{2}$$

The highest strain loss in the concrete lining occurs due to creep as it substantially relaxes the stress imposed by pre-stressing. Under a constant load, the strain loss due to creep, $\varepsilon_{c,creep}$, can be taken approximately up to 40% (Seeber, 1985b). The loss of pressure at the pump is omitted since nowadays the level of the pressure injected can be measured directly at the boreholes. Taking into account the compilation of strain losses, the remaining pre-stress-induced hoop strain at the lining intrados becomes:

$$\varepsilon^{i}_{c,\,p_p} = \varepsilon^{i}_{c,\,p_{inj}} - \varepsilon^{i}_{c,\,\Delta T} - \varepsilon^{i}_{c,\,creep} \tag{3}$$

3. Cracking Process in a Pre-stressed Concrete Lining

The seepage-induced hoop strain at the intrados of an intact concrete lining due to the internal water pressure can be calculated as:

$$\varepsilon^{i}_{c,p_i} = \frac{(p_a - p_i)}{2\,E_c\,(1-\nu_c)} \cdot \left[\frac{1+(r_a/r_i)^2}{(r_a/r_i)^2 - 1} + \frac{\ln(r_a/r_i) + (1-2\nu_c)}{\ln(r_a/r_i)} \right] + \frac{2\,p_F(r_a)}{E_c\,(1-(r_i/r_a)^2)} \tag{4}$$

The mechanical boundary pressure at the lining-grouted zone interface, $p_F(r_a)$, indicating the amount of pressure absorbed by the grouted zone is given by the following equation (Schleiss, 1986b; Simanjuntak et al., 2012):

$$p_F(r_a) = \frac{\dfrac{(1+\nu_g)\,E_c}{(1+\nu_c)\,E_g}\dfrac{(p_g-p_a)}{2(1-\nu_g)} \cdot \left[\dfrac{r_a^{\,2}}{r_g^{\,2}-r_a^{\,2}}\left(1-2\nu_g+(r_g/r_a)^2\right) + (1-2\nu_g)\left(1+\dfrac{1-\nu_g}{\ln(r_g/r_a)}\right)\right] - (p_a-p_i)\left[\dfrac{r_i^{\,2}}{r_a^{\,2}-r_i^{\,2}} + \dfrac{(1-2\nu_c)}{2\ln(r_a/r_i)}\right]}{\dfrac{(1+\nu_g)\,E_c}{(1+\nu_c)\,E_g} \cdot \left[\dfrac{r_a^{\,2}}{r_g^{\,2}-r_a^{\,2}}\left(1-2\nu_g+(r_g/r_a)^2\right)\right] + \left[\dfrac{2\,r_i^{\,2}}{r_a^{\,2}-r_i^{\,2}}(1-\nu_c) + (1-2\nu_c)\right]} \tag{5}$$

A pre-stressed concrete lining will crack as soon as the hoop stress at the lining intrados exceeds the tensile strength of concrete, f_{ctk}. The condition for initial cracks formation in the concrete lining can be expressed as:

$$\varepsilon^{i}_{c,res} = \varepsilon^{i}_{c,p_p} + \varepsilon^{i}_{c,p_i} \geq \frac{f_{ctk}}{E_c} \tag{6}$$

in which $\varepsilon^{i}_{c,pp}$ and $\varepsilon^{i}_{c,pi}$ represent respectively the pre-stress-induced hoop strain and seepage-induced hoop strain at the lining intrados. In this paper, the sign convention for tensile stresses and thus tensile strains is negative.

4. Water Losses and Seepage Pressures

If the concrete lining is intact (Fig. 1a), the water losses, q, from the tunnel situated above the groundwater level can be calculated using (Bouvard, 1975; Bouvard and Niquet, 1980):

$$\frac{p_i}{\rho_w\,g} - \frac{3}{4}r_g = \frac{q}{2\pi\,k_r}\ln\frac{q}{\pi\,k_r\,r_g} + \frac{q}{2\pi}\left[\frac{\ln(r_a/r_i)}{k_c} + \frac{\ln(r_g/r_a)}{k_g}\right] \tag{7}$$

As soon as the lining is cracked, water losses developed throughout the cracked pressure tunnel (Fig. 1b) is governed by the width, $2a$, and the number of cracks in the lining, n, the permeability of the lining between the cracks, k_c, the grouted zone, k_g, and the rock mass, k_r, as well as seepage pressures through these materials. Due to longitudinal cracks, water losses through the lining, q_c, increase and can be estimated as (Schleiss, 1986a):

$$q_c = \frac{(p_i - p_a)\,2\pi\,k_c}{\rho_w\,g\ln(r_a/r_i)} + \frac{(p_i-p_a)\,(2a_i)^3\;n.2.(2a_a/2a_i)^2}{\rho_w\,(r_a-r_i)\,12\,\nu_w\;[1+(2a_a/2a_i)]} \tag{8}$$

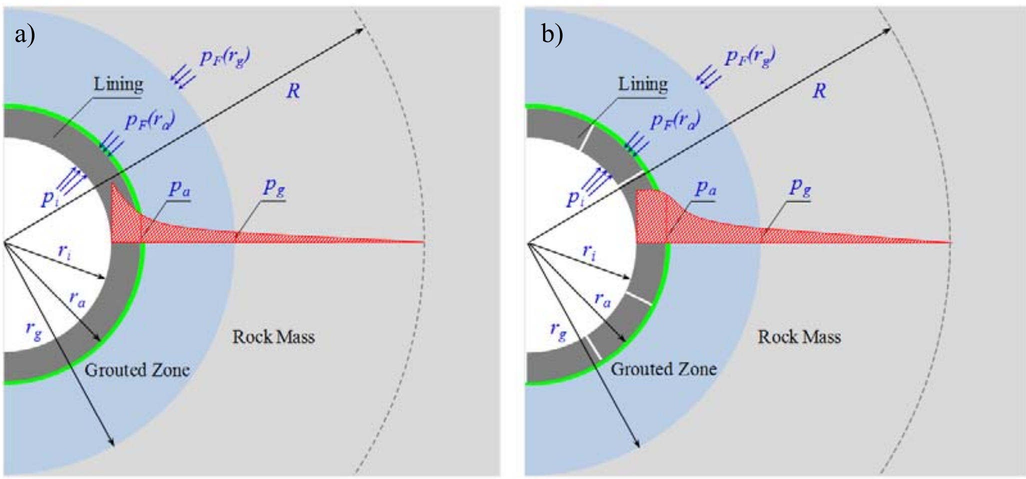

Fig. 2. Distribution of Seepage Pressures through (a) an Intact and (b) a Cracked Concrete Lining (Schleiss, 1986b)

Water losses through the grouted zone, q_g, can be obtained as:

$$q_g = \frac{(p_a - p_g)\, 2\pi\, k_g}{\rho_w\, g\, \ln(r_g / r_a)} \tag{9}$$

Iteratively, the seepage pressure at the grouted zone intrados, p_g, can be calculated using (Bouvard, 1975):

$$\frac{p_g}{\rho_w\, g} - \frac{3}{4} r_g = \frac{q}{2\pi\, k_r} \ln \frac{q}{\pi\, k_r\, r_g} \tag{10}$$

By substituting Eq. (9) to (10), water losses through the grouted zone, q_g, become:

$$q_g = \frac{2\pi\, k_g}{\ln(r_g / r_a)} \left(\frac{p_a}{\rho_w\, g} - \frac{q_g}{2\pi\, k_r} \ln \frac{q_g}{\pi\, k_r\, k_g} - \frac{3}{4} r_g \right) \tag{11}$$

Water losses through the rock mass, q_r, can be calculated as:

$$q_r = \frac{(p_g - p_R)\, 2\pi\, k_r}{\rho_w\, g\, \ln(R / r_g)} \tag{12}$$

in which the reach of seepage flow above the tunnel, R, is given by:

$$R = \left(\frac{q}{\pi\, k_r} \right) \ln(2) \tag{13}$$

Seepage pressures at the extrados of the cracked concrete lining, p_a, can be obtained based on the continuity condition, that is, Eq. (8) is equal to Eq. (9) or (12).

$$p_a = p_i - \frac{\dfrac{\pi\, k_r}{\ln(R / r_g)} \left[p_g - p_R \right]}{\left[\dfrac{\pi\, k_c}{\ln(r_a / r_i)} + \dfrac{g\, (2a_i)^3\, n\, (2a_a / 2a_i)^2}{12\, \nu_w\, (r_a - r_i) \left[1 + (2a_a / 2a_i) \right]} \right]} \tag{14}$$

5. Effect of Cracks in the Lining and the Rock Mass

Due to longitudinal cracks, the concrete lining can only transmit radial stresses to the grouted zone. Depending on the degree of crack openings, $2a_a/2a_i$, the radial stress at the extrados of the cracked lining, $\sigma_r(r_a)$, can be calculated using (Schleiss, 1986a):

$$\sigma_r(r_a) = p_F(r_a) = \frac{(p_a - p_i)}{r_a \left(1 - \dfrac{2a_a}{2a_i} \right)^2 \left(1 + \dfrac{2a_a}{2a_i} \right)} \cdot \left[\left(\frac{2a_a}{2a_i} \right)^2 \left(r_a - \left(2 - \frac{2a_a}{2a_i} \right) r_i \right) + \frac{2a_a}{2a_i} r_i + r_a \left(1 - 2\left(\frac{2a_a}{2a_i} \right) \right) \right] \tag{15}$$

In relation to radial stresses, deformations at the intrados of the grouted zone, $u_g(r_a)$, can be determined using Eq. (16) and this has to correspond to the total width of cracks given by Eq. (17) (Schleiss, 1986b).

$$\begin{aligned} \frac{u_g(r_a)}{r_a} = {} & \frac{(1+\nu_g)(p_a - p_g)}{2 E_g\, (1-\nu_g)} \left[\frac{r_a^2 \left(1 - 2\nu_g + (r_g / r_a)^2 \right)}{r_g^2 - r_a^2} + (1 - 2\nu_g) \left(1 + \frac{(1-\nu_g)}{\ln(r_g / r_a)} \right) \right] - \\ & - \frac{(1+\nu_g)(1-2\nu_g)}{E_g} p_F(r_g) - \frac{r_a^2 (1+\nu_g) \left(1 - 2\nu_g + (r_g / r_a)^2 \right)}{E_g\, (r_g^2 - r_a^2)} \left[p_F(r_g) - \sigma_r(r_a) \right] \end{aligned} \tag{16}$$

$$n\,(2a_a) = u_g\,(r_a)\,2\pi \tag{17}$$

Based on the compatibility condition, that is, the deformation at the extrados of the grouted zone, $u_g(r_g)$, equals to that at the intrados of the rock mass, $u_r(r_g)$, the unknown mechanical boundary stress at the grouted zone-rock interface, $p_F(r_g)$, can be calculated as (Schleiss, 1986b; Simanjuntak et al., 2012):

$$p_F(r_g) = \frac{\dfrac{(1+\nu_r)\,E_g}{(1+\nu_g)\,E_r}\,\dfrac{(p_R-p_g)}{2(1-\nu_r)}\cdot\left[\dfrac{r_g^{\,2}}{R^2-r_g^{\,2}}\left(1-2\nu_r+(R/r_g)^2\right)+(1-2\nu_r)\left(1+\dfrac{1-\nu_r}{\ln(R/r_g)}\right)\right]-(p_g-p_a)\left[\dfrac{r_a^{\,2}}{r_g^{\,2}-r_a^{\,2}}+\dfrac{(1-2\nu_g)}{2\ln(r_g/r_a)}\right]}{\dfrac{(1+\nu_r)\,E_g}{(1+\nu_g)\,E_r}\cdot\left[\dfrac{r_g^{\,2}}{R^2-r_g^{\,2}}\left(1-2\nu_r+(R/r_g)^2\right)\right]+\left[\dfrac{2\,r_a^{\,2}}{r_g^{\,2}-r_a^{\,2}}(1-\nu_g)+(1-2\nu_g)\right]} \tag{18}$$

The crack width at the lining intrados, $2a_i$, can thus be determined using (Schleiss, 1986a):

$$(2a_i) = (2a_a)\,(r_a/r_i) \tag{19}$$

6. Calculation Procedure

The following step-by-step calculation procedure is proposed so as to predict the distribution of seepage pressures and water losses around the passive pre-stressed concrete-lined pressure tunnel.

(A) As a result of pre-stressing works, calculate the pre-stress-induced hoop strains at the lining intrados, $\varepsilon^{i}_{c,pp}$, using Eq. (3) by considering thermal and creep losses.

(B) Assume the internal water pressure, p_i, to be somewhat higher than the pre-stressing level and calculate the corresponding water losses, q, directly from Eq. (7).

(C) Determine seepage pressures acting at the lining extrados, p_a, and at the grouted zone-rock interface, p_g, using Eqs. (9) and (10). At the same time, compute the corresponding seepage-induced hoop strains at the lining intrados, $\varepsilon^{i}_{c,pi}$, from Eq. (4) by considering stresses transmitted to the grouted zone, $p_F(r_a)$, calculated from Eq. (5).

(D) Increase the internal water pressure and investigate the crack status in the lining according to the condition given by Eq. (6). As soon as the permissible tensile strength of concrete is exceeded, longitudinal cracks in the lining will occur.

(E) Assume the new seepage pressure at the lining extrados, p_a, to a value that is near to the internal water pressure, p_i.

(F) Assume the water losses from the cracked concrete-lined pressure tunnel, q, to a value that is higher than the preceding value which was calculated using Eq. (7). Recalculate the corresponding seepage pressures developed in the grouted zone, p_g, and the rock mass, p_R, using Eqs. (10) and (12) by considering Eq. (13).

(G) Determine stresses transmitted to the grouted zone, $p_F(r_a)$, and to the rock mass, $p_F(r_g)$, from Eqs. (15) and (18) respectively by taking into account Eq. (19).

(H) Calculate the deformation in the grouted zone, $u_g(r_a)$, using Eq. (16). For a given number of cracks, n, the width of cracks at the lining extrados, $2a_a$, and intrados, $2a_i$, can therefore be established using Eqs. (17) and (19).

(I) Once the crack openings at the intrados and extrados of the lining are obtained, control the seepage pressure at the lining extrados, p_a, using Eq. (14) and the corresponding water losses, q_c, using Eq. (8). Repeat the calculation steps (E) to (H) until the seepage pressure at the lining extrados, p_a, and the actual water losses throughout the system, q, remain constant.

7. Calculation Example

In this section, the performance of a passive pre-stressed pressure tunnel taking into account the lining cracking process is discussed in an example. It is a circular tunnel with an internal diameter of 5.6 m and lined by a concrete lining having a nominal thickness of 35 cm. The pressure tunnel is situated above the groundwater level and embedded in the rock mass characterized by a uniform in-situ stress, σ_o = 30 MPa and permeability, $k_r = 10^{-6}$ m/s. For simplifications, the rock mass is treated as a homogenous, isotropic, porous and elastic material. The consolidation grouting the surrounding rock mass is performed up to an external radius, r_g, of 5 m. Parameters used in the analysis are summarized in Table 1.

Table 1. Material Properties

Rock Mass		Grouted Zone		Concrete C25/30			
E_r (GPa)	v_r	E_g (GPa)	v_g	f_{ctk} (MPa)	f_{ck} (MPa)	E_c (GPa)	v_c
15	0.20	15	0.20	22.5	1.8	30.5	0.20

7.1 Maximum Bearing Capacity of Concrete-Lined Pressure Tunnels

In this paper, the pre-stress grout, p_{inj}, of 15 bar was used as an example. Assuming the temperature change and the pre-stress loss due to creep are respectively of 10^{o} C and 30%, the pre-stress-induced hoop strains at the lining intrados, $\varepsilon^{i}_{c,pp}$, was obtained as 0.176‰.

The permeability of an intact concrete lining in a pressure tunnel is normally below 10^{-8} m/s (Schleiss, 1997a). According to Portland Cement Association (1979), the permeability of mature, good quality concrete without any fissures and construction irregularities is about 10^{-12} m/s. For various values of the lining permeability, the maximum bearing capacity of concrete-lined pressure tunnels is presented in Fig. 3a. The corresponding seepage pressure and water losses built up behind the lining are given in Figs. 3b and c respectively.

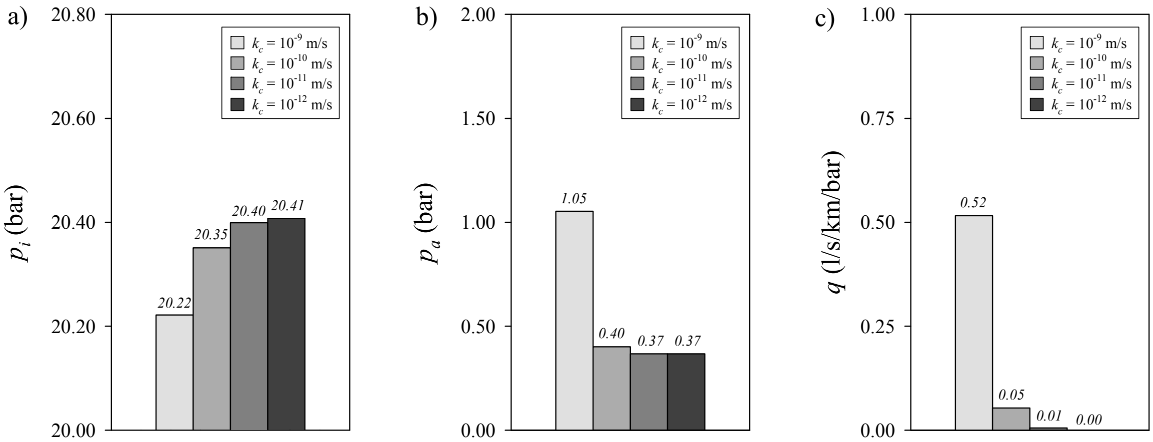

Fig. 3. Effect of the Lining Permeability on the (a) Bearing Capacity (b) Pressure at the Lining Extrados and (c) Water Losses

For cases where the concrete lining is intact and nearly impervious, a high bearing capacity of pressure tunnels can be expected. The more pervious the lining is, the lower the bearing capacity of pressure tunnels will become to withstand the load exerted by the internal water pressure (Fig. 3a). This observation is in a good agreement with that presented in Schleiss (1986b). Due to the fact that the inner surface of concrete-lined pressure tunnels is not impervious, the water from the tunnel will seep into the rock mass inducing seepage pressures, which act not only in the concrete lining, but also in the rock mass (Fig. 3b). Seepage pressures cannot be overlooked in the design since this parameter determines the maximum bearing capacity of concrete-lined pressure tunnels and thus the maximum water head inside the tunnel.

The influence of the concrete permeability on water losses is presented in Fig. 3c. If the concrete lining is intact and nearly impervious, i.e. $k_c \leq 10^{-10}$ m/s, water losses can be negligible. For cases where the lining is pervious, the rate of water losses in the order of 1 l/s/km/bar (Schleiss, 1988) should not be exceeded so as to avoid the washing out of the joint fillings in the rock mass. When the safety of the tunnel is not put at risk, the water losses in the order of 2 l/s/km/bar (Marence, 2008) can still be tolerated.

7.2 Effect of the Grouted Zone Permeability

Once the tensile strength of the concrete is exceeded so that the lining is cracked, the internal water pressure will act partly or entirely at the lining extrados. As a consequence, high local water losses occur around the opening, which may result in washing out of the joint fillings and instability of the tunnel.

If cracks in the lining are inevitable, the crack widths have to be limited so as to prevent the gradual process of erosion of fine rock materials behind the lining. Such erosion should not occur if the width of the cracks is less than 0.3 mm (Schleiss, 1997b).

For passive pre-stressed concrete-lined pressure tunnels, the application of consolidation grouting is a physical requirement. Besides to reinstate the mechanical properties of the rock mass around the tunnel and to provide a continuous load transfer between the lining and the rock mass, the purpose of this grouting has been recognized in many years as a technique to reduce the water losses and refine the cracks, if any (Simanjuntak et al., 2012). The permeability of the rock mass can be reduced up to 1 Lugeon or approximately to 10^{-7} m/s with cement grouting (Schleiss, 1997a). With stable grout, the permeability of up to 0.1 Lugeon can be achieved (Barton et al., 2001; Fernandez, 1994). Nowadays, using micro-cements in combination with additives such as micro-silica and plasticizers, the permeability lower than 0.1 Lugeon are implied (Barton, 2004), however this creates a large spectrum of applications (Vigl and Gerstner, 2009), which eventually leads to a more expensive construction costs.

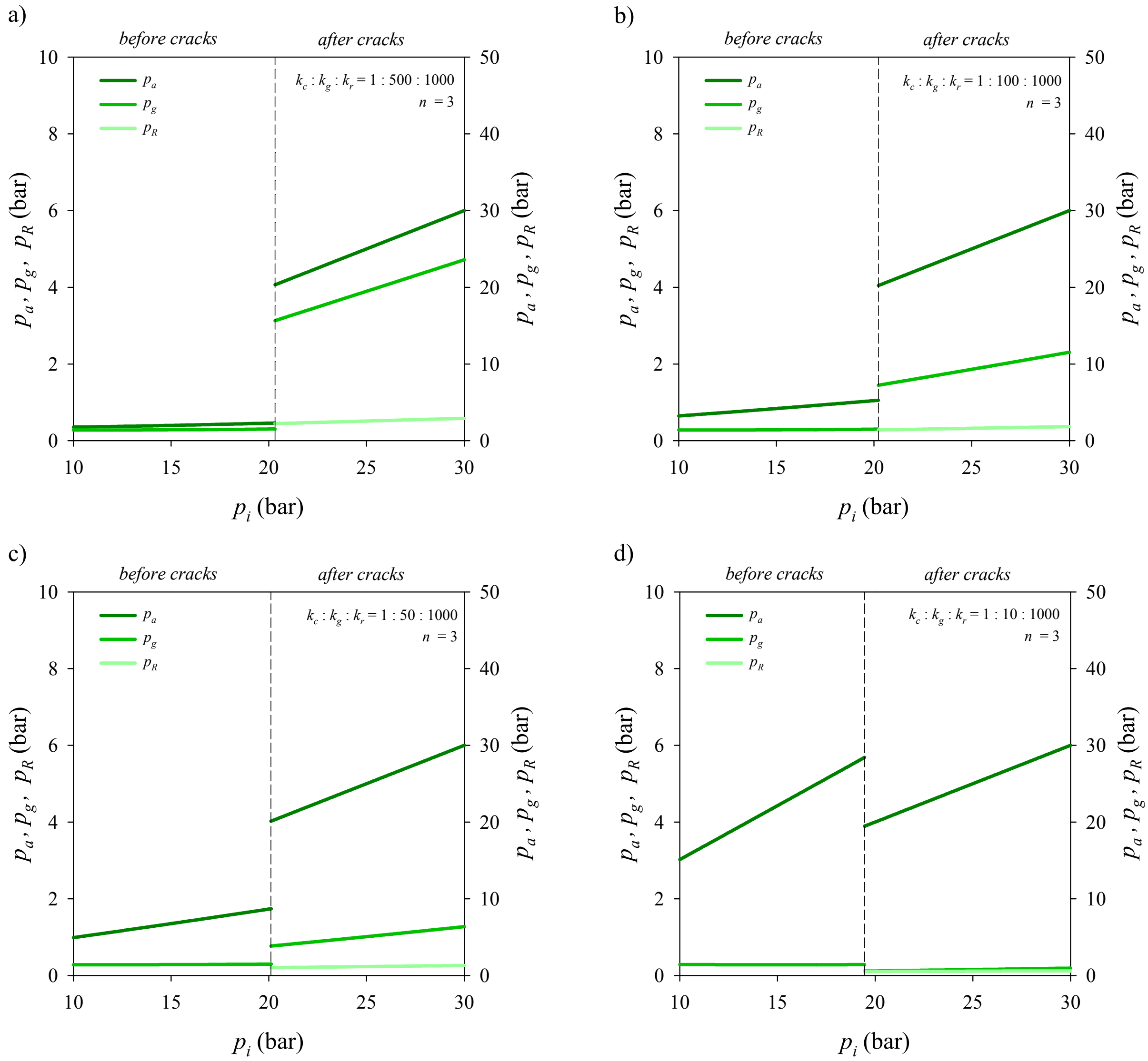

Fig. 4. Effect of the Grouted Zone on Seepage Pressures

As shown in Fig. 4, the less permeable the grouted zone is, the higher the seepage pressure behind the intact lining, p_a, will become and the lower the seepage pressure, p_g, is in the rock mass. Consistent with that presented in Simanjuntak et al. (2012), reducing the permeability of the grouted zone does not necessarily mean to increase the bearing capacity of pressure tunnels (Fig. 5a) since the bearing capacity is governed by mechanical boundary pressures (Figs. 5b and c). If the lining is cracked, high seepage pressures occurring in the rock mass, p_g, and p_R, can still be reduced with consolidation grouting.

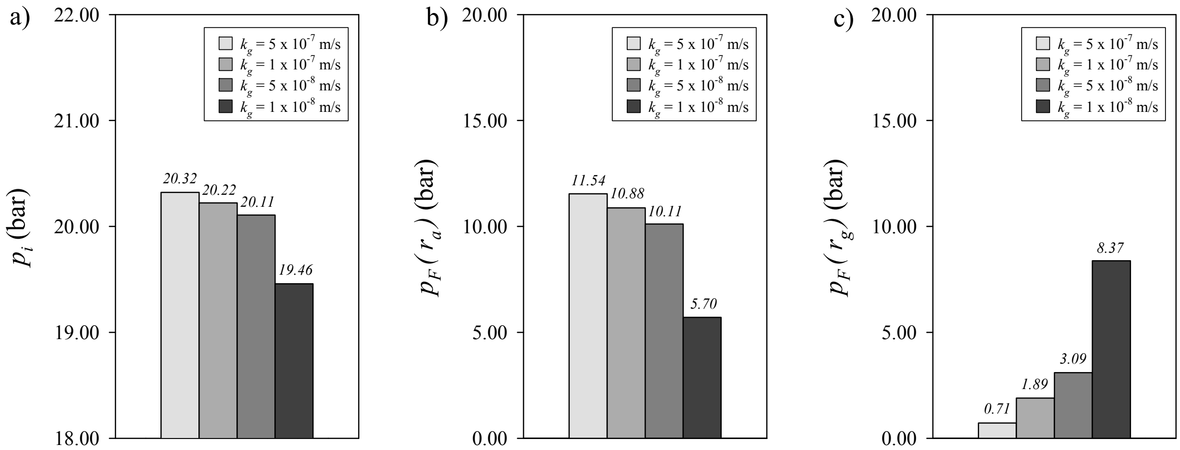

Fig. 5. Effect of the Grouted Zone on the (a) Bearing Capacity and (b, c) Mechanical Boundary Pressures

The estimation of water losses from the tunnel, q, for various value of the grouted zone permeability, k_g, is presented in Fig. 6. As long as the lining is intact, water losses are still within the permissible values. If the lining is cracked, water losses increase so much that they can potentially result in washing out of the joint fillings and instability of the tunnel against hydro-jacking if natural rock stresses are low.

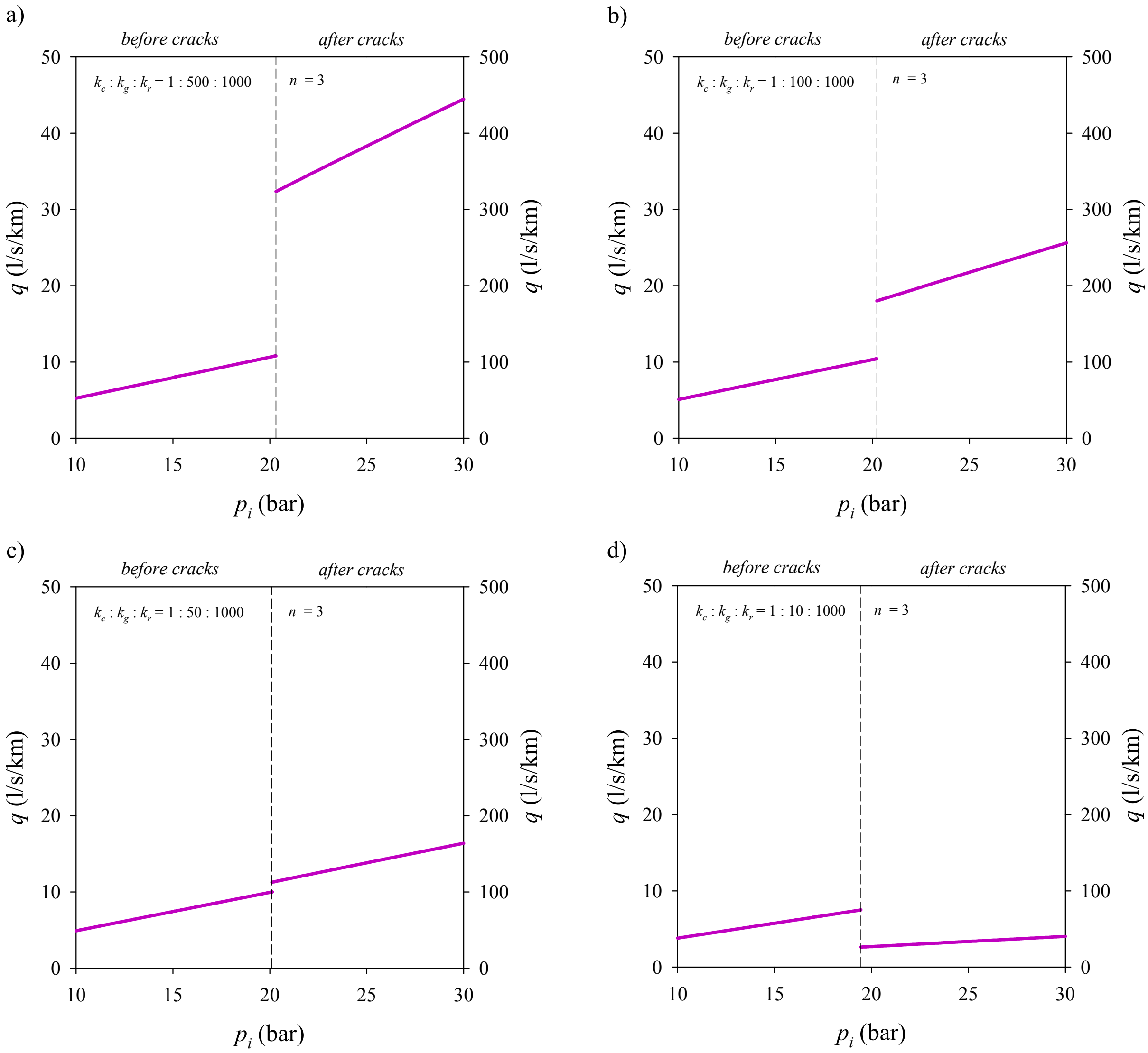

Fig. 6. Effect of the Grouted Zone on Water Losses

In tunnelling, the economical depth of grouting, r_g, is about 1 to 2 radius of the tunnel (Schleiss, 1986b). In the case of high rock mass permeability, water losses from the tunnel can still be too high if the permeability of the grouted zone, k_c, is greater than 10^{-8} m/s (Fig. 6). Herein, reducing the rock mass permeability by improving the grouting quality is more effective than lengthening the grouting depth so as to avoid high seepage pressures and water losses.

If cracks in the lining are inevitable but the safety of the tunnel is not put at risk, excessive water losses can still be avoided provided the permeability of the grouted zone, k_g, is no less than 10^{-8} m/s, which can be achieved in practice using waterproofing grouting. With regard to economical aspects, concrete linings with plastic sheeting, such as polyvinyl (PVC) or polyethylene (PE) (Seeber, 1984) are required so as to avoid water losses.

7.3 Effect of Cracks on Mechanical Boundary Pressures

For passive pre-stressed concrete-lined pressure tunnels embedded in an intact rock, favourite locations of longitudinal cracks in the lining are at the tunnel roof and at the transition floor-wall. The number of cracks, n, according to this pattern is 3 (Bouvard, 1975). A few large cracks, i.e. $n < 10$, can be expected in the lining embedded in fractured rock masses.

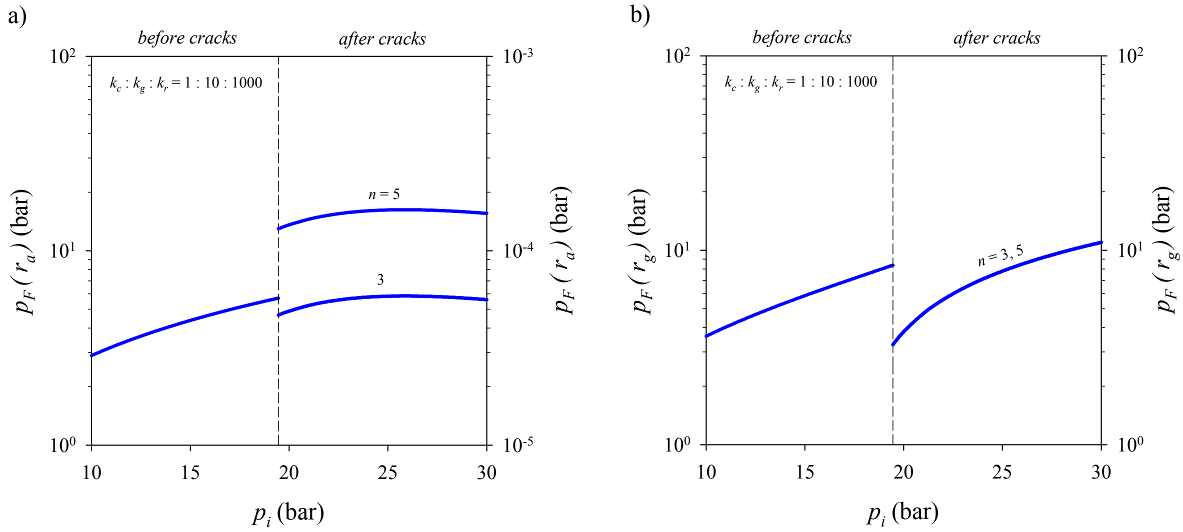

Fig. 7. Mechanical Boundary Pressures at (a) Lining-Grouted Zone, and (b) Grouted Zone-Rock Mass Interfaces

The effect of the number of cracks in the lining on the mechanical boundary pressures is presented in Fig. 7. While Fig. 7a depicts the effect of the number of cracks on the mechanical boundary pressure at the lining-grouted zone interface, $p_F(r_a)$, Fig. 7b illustrates its influence on the mechanical boundary pressure at the grouted zone-rock mass interface, $p_F(r_g)$.

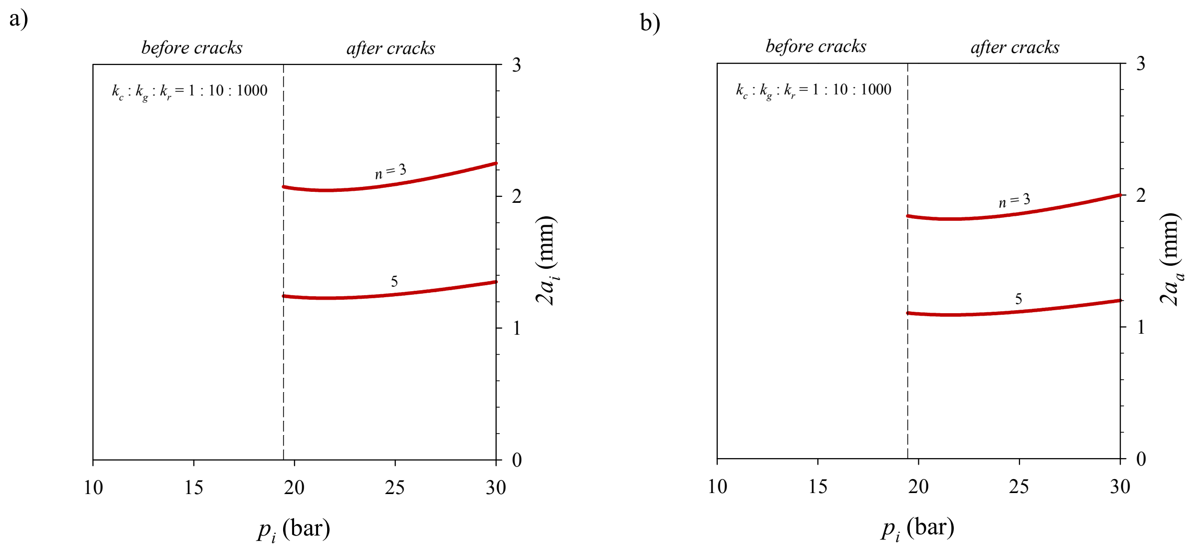

Fig. 8. Crack Widths at the (a) Intrados and (b) Extrados of the Concrete Lining

When the lining is cracked, the internal water pressure is shifted from the intrados to the extrados of the cracked lining. Consequently, the mechanical boundary pressure at the lining-grouted zone interface, $p_F(r_a)$, decreases considerably to a value near to zero (Fig. 7a). The portion of the pressure transmitted from the grouted zone to the rock mass will be depending on the grouted zone and rock mass properties. The tighter the grouted zone is, the poorer the rock mass quality that means and the lower the mechanical boundary pressure at the grouted zone-rock mass, $p_F(r_g)$, will become (Fig. 7b).

Fig. 8 illustrates the effect of the number of cracks on cracks widths. The discrepancy of crack widths between those at the lining intrados, $2a_i$, and those at intrados, $2a_a$, decreases as the number of cracks, n, increases. It has to be noted that the width of the cracks more than 0.3 mm can induce deleterious effects on the joint fillings of the rock mass, and therefore the geometry of cracks must be limited. With the passive pre-stressing technique, this goal is nevertheless difficult to achieve. In this regard, the combination of consolidation grouting with the economical reinforcement (Schleiss, 1997b) or the active pre-stressing technique (Matt et al., 1978) can be suggested as design alternatives.

Principally, if the longitudinal cracks in the concrete lining can be accepted and as long as the pressure tunnel is embedded in the hard rock where the stability of the tunnel against hydro-jacking can be preserved, no pre-stressing of the concrete lining is necessary. However, water losses and seepage pressure behind the lining must be expected. Depending on the rock mass permeability, the use of plastic sheeting or economical grouting can be applied so as to limit the development of seepage pressure and water losses.

8. Conclusions

The method to predict the maximum bearing capacity of passive pre-stressed concrete-lined pressure tunnels has been presented in this paper. It accounts for the effect of longitudinal cracks on the development of seepage pressures built up behind the lining. The following zones are considered in the analysis: circular concrete lining, grouted zone and rock mass.

The study confirms the relevance of seepage pressures on the bearing capacity of passive pre-stressed concrete-lined pressure tunnels. Seepage pressure cannot be overlooked in the design since their flow into the rock mass exists regardless of cracks in the lining, which leads to an additional relief of the load on the lining. Once cracks in the lining occur, the internal water pressure is shifted to the lining extrados. High local seepage pressures as well as water losses may take place, rendering the overall stability of the pressure tunnel depends exclusively on the bearing capacity of the rock mass.

As long as the tunnel stability against hydro-jacking can be preserved by the rock mass, provided by adequate strength and/or overburden, detrimental effects of cracks as a result of excessive water losses can be avoided by applying waterproofing grouting or economical plastic sheeting. Otherwise, no cracks are allowed in the lining if the passive pre-stressing technique needs to be applied so as to ensure the long-term stability of pressure tunnels. The governing design criteria for passive pre-stressed concrete-lined pressure tunnels are thus: avoiding cracks in the lining, limiting water losses and ensuring the bearing capacity of the rock mass.

The step-by-step calculation procedure proposed in this paper is capable of predicting the distribution of seepage pressures and water losses around passive pre-stressed concrete-lined pressure tunnels. Even though its applicability is restricted to the assumptions given in this paper, it can provide realistic insights for more elaborate numerical studies regarding the effect of longitudinal cracks on the performance of passive pre-stressed pressure tunnels embedded in anisotropic rock mass deformability and permeability.

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Notations

$2a_i$	crack width at the lining intrados	[mm]
$2a_a$	crack width at the lining extrados	[mm]
E_c	elasticity modulus of the lining	[GPa]
E_g	elasticity modulus of the grouted zone	[GPa]
E_r	elasticity modulus of the rock mass	[GPa]
f_{ctk}	compressive strength of the concrete	[MPa]
f_{ck}	tensile strength of the concrete	[MPa]
g	gravity acceleration	[m/s^2]
k_c	permeability of the intact lining	[m/s]
k_g	permeability of the grouted zone	[m/s]
k_r	permeability of the rock mass	[m/s]
n	number of longitudinal cracks	[-]
p_a	seepage pressure at the lining extrados	[bar]
p_i	internal water pressure	[bar]
p_g	seepage pressure at the grouted zone extrados	[bar]
p_R	seepage pressure in the rock mass	[bar]
p_{inj}	pre-stress grout	[bar]
$p_F(r_a)$	mechanical boundary pressure between the lining and the grouted zone	[bar]
$p_F(r_g)$	mechanical boundary pressure between the grouted zone and the rock mass	[bar]

$p_F(R)$	mechanical boundary pressure between the saturated and the unsaturated rock mass	[bar]
q_c	water losses through the intact lining	[m^2/s]
q_g	water losses through the grouted zone	[m^2/s]
q_r	water losses through the rock mass	[m^2/s]
r_a	external radius of the lining	[m]
r_i	internal radius of the lining	[m]
r_g	radius of the grouted zone	[m]
R	external radius of the rock mass affected by the seepage	[m]
$u_g(r_a)$	deformation at the lining-grouted zone interface	[mm]
α_T	thermal coefficient	[1/^{o}C]
ΔT	temperature change	[^{o}C]
$\varepsilon^i_{c,\Delta T}$	strain losses at the lining intrados due to thermal cooling	[-]
$\varepsilon^i_{c,\,creep}$	strain losses at the lining intrados due to creep	[-]
$\varepsilon^i_{c,\,pp}$	pre-stress induced hoop strain at the lining intrados	[-]
$\varepsilon^i_{c,\,pi}$	seepage induced hoop strain at the lining intrados	[-]
$\varepsilon^i_{c,\,res}$	residual hoop strain at the lining intrados	[-]
v_c	Poisson's ratio of the concrete	[-]
v_g	Poisson's ratio of the grouted zone	[-]
v_r	Poisson's ratio of the rock	[-]
v_w	kinematic viscosity of water	[m^2/s]
ρ_w	water density	[kg/m^3]
$\sigma_r(r_a)$	radial stress at the extrados of the cracked lining	[MPa]

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