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Numerical modeling of coupled structural and hydraulic interactions in tunnel linings

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Abstract. Tunnels are generally constructed below the ground water table, which produces a long-term interaction between the tunnel lining and the surrounding geo-materials. Thus, in conjunction with tunnel design, the presence of water may require a number of considerations such as: leakage and water load. It has been reported that deterioration of a drainage system of tunnels is one of the main factors governing the long-term hydraulic and structural lining-ground interaction. Therefore, the design procedure of an underwater tunnel should address any detrimental effects associated with this interaction. In this paper an attempt to identify the coupled structural and hydraulic interaction between the lining and the ground was made using a numerical method. A main concern was given to local hindrance of flow into tunnels. Six cases of local deterioration of a drainage system were considered to investigate the effects of deterioration on tunnels. It is revealed that hindrance of flow increased pore-water pressure on the deteriorated areas, and caused detrimental effects on the lining structures. The analysis results were compared with those from fully permeable and impermeable linings.

Keywords: tunnel lining; hydraulic deterioration; coupled structural and hydraulic interaction.

1. Introduction

Research on soil-structure interaction generally considers nonlinear static behavior (Jahromi *et al.* 2008), or short-term behavior under earthquakes (Lehmann 2005). Meanwhile fluid-structure-soil interaction problems are mainly concerned fluid in pipelines or storage tanks under dynamic loadings (Cassidy 2006, Kim *et al*. 2002). Those studies rarely include fluid (water) in soils. Geotechnical systems such as tunnels could, however generate the long-term ground water-structure-soil interaction under mechanically static conditions.

Tunneling below the ground water table causes seepage into the tunnel and often produces a long-term interaction between the tunnel and the surrounding geo-material. In conjunction with tunnel design, the presence of water adds a number of problems such as: leakage and additional pore water pressure. Pore-water pressure is particularly important in tunnel lining design (Bobet 2003). To reduce pore-water pressure, tunnels are often designed to act as drains to reduce pore-water pressure by adopting drainage systems.

Several cases of lining failure caused by water pressure were reported (Lee *et al.* 1996, Ferreira 1995), and Fig. 1(a) shows an example of a damaged lining. It is also reported that one of the most

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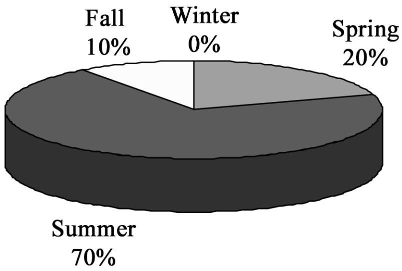


(a) **Co**ncrete lining failure (Shin et al. 2005) (b) **Oc**currence of lining failure (KISTEC 2007)

Fig. 1 Lining damages relevant to water pressures

critical facts causing such failures is hydraulic deterioration of a drainage system. Recent research showed that the lining damages mostly occur during rainy seasons when the ground water level is high. Fig. 1(b) supports the relevance of water pressure (KISTEC 2007). It is also identified that deterioration of drainage systems occurs due to squeezing force during concrete placement and /or clogging the filters, hinders flow into tunnels by reducing permeability, and consequently develop pore-water pressure which causes additional stresses on the tunnel lining system (Reddi *et al.* 2000, Lee *et al.* 2002). Thus, the behavior can be termed the coupled structural and hydraulic interaction.

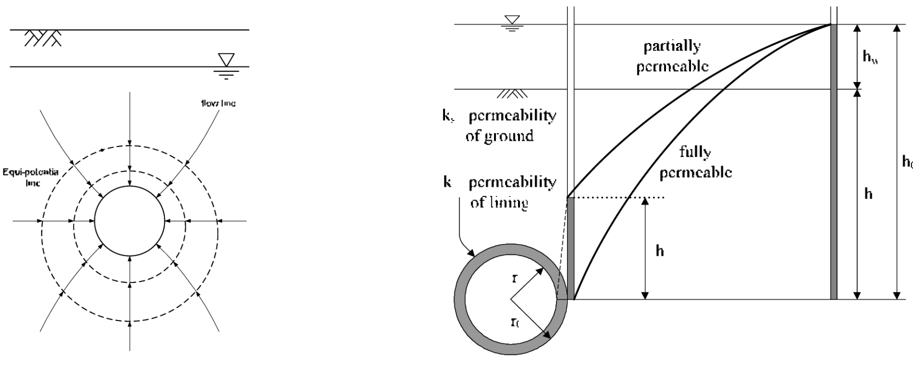
Deterioration of a drainage system would occur throughout the life time of a tunnel, and frequent be found particularly in aged-tunnels. The tunnel lining design guide (BTS and ICE 2004) indicates that the design procedures should include provision for such time dependency. Therefore the design procedure of a tunnel below ground water table has to address any detrimental effects associated with the deterioration of the drainage system. Unfortunately, however, the complicated structural and hydraulic boundary conditions of the problem and the long time period required to measure the tunnel and ground behavior are the main difficulties in identifying the structural and hydraulic interaction.

Numerical methods can provide a useful tool for investigating the problems for their flexibility and ability to model complex boundary conditions (Shin *et al.* 2002). In this paper, the structural and hydraulic behavior of a concrete lining and a geo-materials was investigated by employing a coupled finite element method. Particular concerns were given to the local deterioration of drainage systems, as in reality deterioration would not occur all over the lining, but in certain parts of linings which have structural defects, or cavities in the lining.

2. Finite element modeling

Modelling of a tunnel below ground water table needs to consider both structural and hydraulic facets. Fig. 2(a) shows the flow-net for an underwater tunnel with fully permeable hydraulic boundary conditions which have no pore-water pressures on the lining. Installation of lining (or deterioration of drainage system), however may hinder flow into a tunnel as shown in Fig. 2(b), modify hydraulic boundary conditions and generate additional water heads on the linings.

Modeling of such a coupled structural and hydraulic interaction problem requires governing equations combining displacement (or effective stress) and pore-water pressure. This can be achieved by introducing the principle of effective stress in geotechnical engineering.



(a) Flow adjacent to an unlined tunnel (b) Development of water head on the lining

Fig. 2 Coupled structural and hydraulic interaction between the tunnel and the ground

$$\sigma'_{ij} = \sigma_{ij} - p\delta_{ij} \tag{1}$$

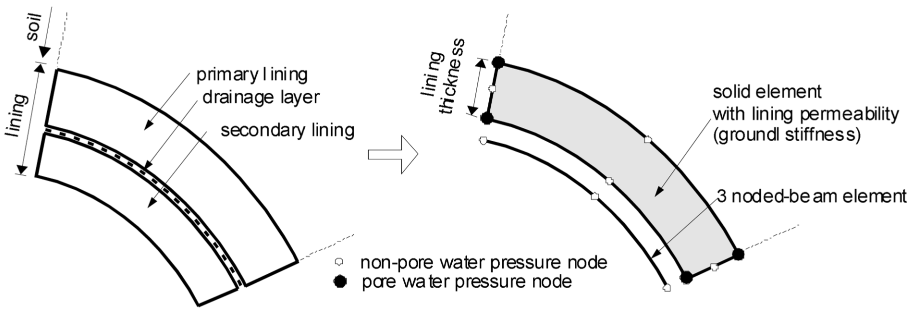
where σ_{ij} and σ'_{ij} are the total and the effective stresses respectively, p is the pore-water pressure and δ_{ij} is a Kronecker delta.

The coupled finite element equations considering deformation and pore-water pressure can be formulated using Biot's theory (1941). In this paper finite element schemes proposed by Booker and Small (1975) are adopted, which can be written as

$$\begin{bmatrix} K_G & L_G \\ L_G^T & -\beta \cdot \Delta t \cdot \Phi_G \end{bmatrix} \begin{Bmatrix} \Delta d_{nG} \\ \Delta p_{nG}{}^{t_2} \end{Bmatrix} = \begin{Bmatrix} \Delta R_G \\ \Phi_G \cdot p_{nG}{}^{t_1} \cdot \Delta t \end{Bmatrix} \tag{2}$$

where K_G is the average global stiffness matrix over the time interval (t_1, t_2), R_G is the right side load vector, L_G is the global coupling matrix, Φ_G is the global flux matrix, d_n and p_n are the global vectors of the unknown nodal displacement and pore pressure respectively, Δt denotes time interval $(t_1 - t_2)$. β is the numerical integration parameter.

A typical lining structure of a bored tunnel (i.e., New Austrian Tunnelling Method: NATM) is shown in Fig. 3(a). To model the coupled structural and hydraulic behavior, a lining with a finite



(a) Typical NATM lining structure (b) Lining model for coupled behavior

Fig. 3 Modelling the coupled hydraulic and structural lining behavior permeability was considered through the introduction of a special scheme combining structural beam element, which give direct solutions for lining distortions, forces and moments, and thin quadrilateral solid elements, which can have a prescribed permeability of concrete lining. The arrangement is shown in Fig. 3(b). The clogging phenomenon can be represented numerically by reducing the permeability of the lining.

3. Finite element analysis

A model tunnel to be analyzed in this study is shown in Fig. 4. Ground profiles and material parameters are also presented in Fig. 4.

3.1 Mechanical models

The ground is modelled by eight-noded isoparametric solid continuum elements. The pre-yield behavior of the decomposed granite soil is represented by the nonlinear elastic small strain model

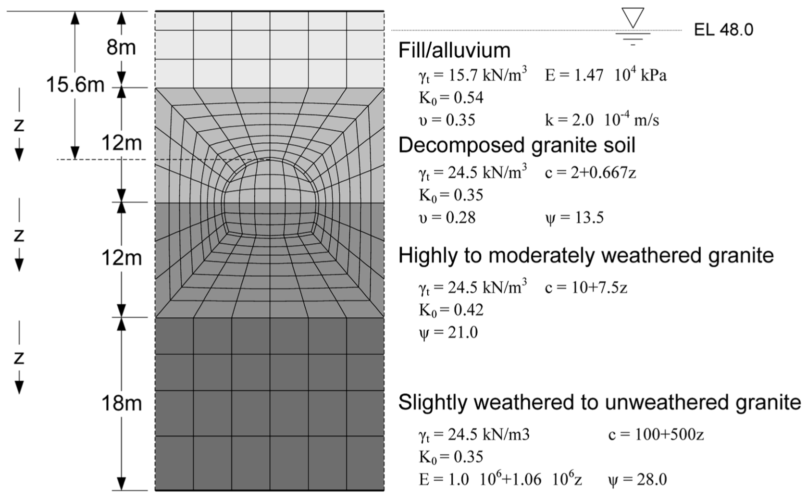


Fig. 4 Model tunnel, ground profiles and material parameters

Table 1 Material parameters

Small strain non-linear elastic parameters for decomposed granite soil								
Shear modulus parameters	C_1 1515	C_2 1485	C_3 : % 2×10^{-4}	c_1 0.955	c_2 0.818	$E_{d\,\min}$: % 9.0×10^{-3}	$E_{d\,\max}$: % 0.35	$G_{\min}$: kPa 9706
Bulk modulus parameters	C_4 475	C_5 465	C_6 2×10^{-4}	c_3 0.848	c_4 0.872	$\varepsilon_{v\,\min}$: % 5.0×10^{-3}	$\varepsilon_{v\,\max}$: % 0.5	$K_{\min}$: kPa 6438
Material parameters for linings								
cross-sectional area: 0.268 m^2/m second moment of area: 0.0016 m^4/m					Young's modulus(E): 2.0×10^7 kPa Poisson's ratio (ν) : 0.2			

proposed by Jardine *et al.* (1986), of which modified can be found in Potts and Zdravkovic (1999).

$$\frac{3G}{p'} = C_1 + C_2\cos\left(c_1\left[\log\frac{E_d}{C_3}\right]^{c_2}\right) \quad \text{and} \quad \frac{K}{p'} = C_4 + C_5\cos\left(c_3\left[\log\frac{|\varepsilon_v|}{C_6}\right]^{c_4}\right) \tag{3}$$

The relevant parameters for decomposed granites are given in Table 1, where G is the tangent shear modulus, K is the tangent bulk modulus, E_d is the deviatoric strain, ε_v is the volumetric strain and C_1, C_2, C_3, C_4, C_5, C_6, c_1, c_2, c_3, c_4 are all coefficients. In this model, the variation of tangent shear and bulk moduli are represented by the periodic logarithmic functions. The coefficients were obtained by fitting the bender element and triaxial test data of the ground layers in the range of strain. A linear elastic model is used to represent the pre-yield behavior for other ground materials. The Mohr-Coulomb model is used to represent post-yield behavior of ground materials. The tunnel lining is modelled by the 3-noded linear elastic beam elements based on Mindlin beam theory (Day and Potts 1990). The material parameters are listed on Table 1.

3.2 Permeability models

Modelling of flow behavior requires prescribing the permeability model as described in Eq. (2). The flow behavior of the decomposed granite was modelled using the non-linear permeability model proposed by Vaughan (1989), where the permeability, k varies exponentially with mean effective stress, p'.

$$k = k_o\exp(-B\cdot p') \tag{4}$$

where k_o is the coefficient of permeability at $p' = 0$, where p' is the mean effective stress $(= (\sigma_1' + \sigma_2' + \sigma_3')/3)$ and B is the material constant.

According to Neville (1995), the permeability of concrete (or shotcrete) typically varies from 10^{-10} to 10^{-12} m/s. In reality however, the major paths of water intrusion into a concrete tunnel lining are cracks and construction joints. Thus, seepage into a tunnel lining is influenced by the mass permeability rather than by that of the concrete itself. Consequently, the evaluation of lining permeability is very difficult and has rarely been reported.

The rate of flow into a tunnel will be dependent on the relative permeability of the lining, k_l, to the adjacent soil, k_s (Shin *et al.* 2005). Therefore in this study, only the relative permeability, k_l/k_s is considered and the case where the ratio, $k_l/k_s < 1.0$ is mainly concerned.

3.3 Initial and boundary conditions

The initial stresses prior to tunnel excavation were defined by the bulk unit weight of the soil (γ_t) and the coefficients of earth pressure at rest (K_o). Initial pore-water pressures were assumed to be hydrostatic. Initial conditions for the long-term analysis are obtained from the construction analyses which include excavation and lining installation.

Although the hydraulic boundary conditions on the ground surface are governed by the climate and the hydro-geophysical environment, the effect of stress path reversals is not accounted for in this study. It was assumed that the phreatic surface was maintained at a depth of 2.5 m below the ground surface throughout the analysis. On the right and left hand sides of the model vertical boundary the pore-water pressures were assumed to remain at their initial hydrostatic values.

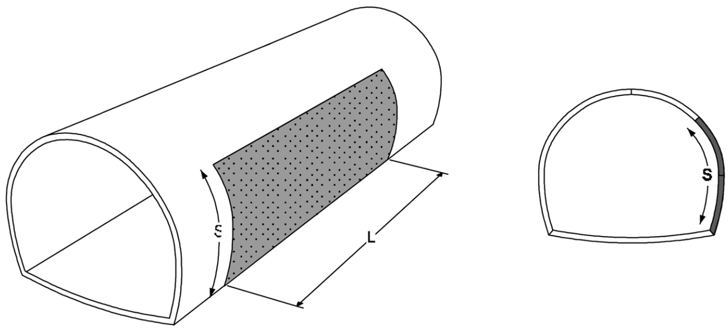
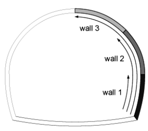
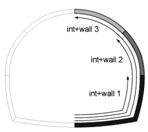


Fig. 5 Local deterioration of a drainage system

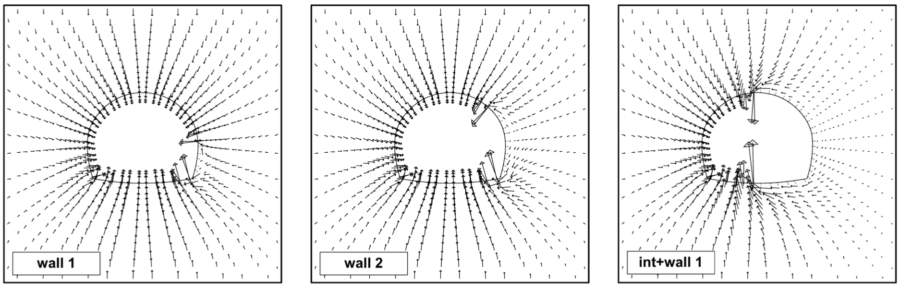
Table 2 Analysis cases (h.b.c: hydraulic boundary condition; k_f: permeability of filter)

Cases	Deterioration (hindrance of flow)		Permeability (deteriorated area)	h.b.c at the lining	Symbol
Whole hindrance of flow	Fully permeable Impermeable		$k_l = k_s = k_f$ -	$p = 0$ $q = 0$	fully permeable impermeable
Local hindrance of flow	Wall		- deteriorated area: $k_l / k_s = 1.0$ $k_f / k_l = 0.001$ - other area: $k_l = k_s = k_f = 1.0$	$p = 0$	wall 1 wall 2 wall 3
	Invert + wall				int +wall 1 int +wall 2 int +wall 3

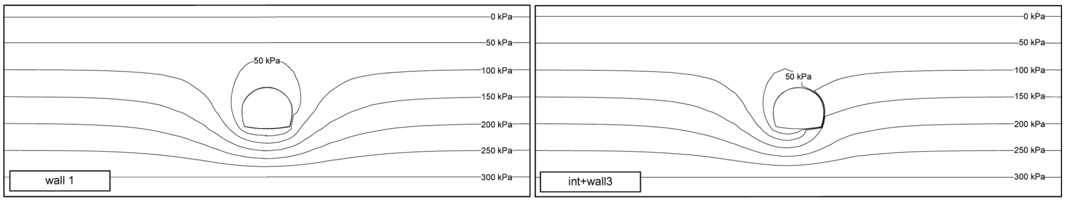
To model the local deterioration, i.e. local hindrance of flow into a tunnel as shown in Fig. 5 is assumed. Deterioration of filter drainage system is represented by reducing the permeability of the deteriorated zone, and clogging of drain hole is simulated by prescribing zero flow rate, as $q = 0$.

3.4 Analysis cases

The effect of whole lining deterioration was investigated by Shin *et al.* (2005) with emphasis on ground behavior. However, field investigation shows that lining damages generally occur in certain parts of linings which have structural defects such as cracks or cavities. In this study local deterioration of drainage system including clogging of drain holes is mainly concerned. Local deterioration is modeled in two dimensions by assuming the deteriorated length, L in Fig. 5 is sufficiently greater than the total length of tunnel periphery. Three cases of wall deterioration and three cases of invert and wall deterioration are considered. Table 2 lists analysis cases. Fully permeable and impermeable cases are also considered to compare the results with those from the local deterioration cases.



(a) Seepage velocity vectors



(b) Pore-water pressure distribution

Fig. 6 Flow behavior due to local deterioration

4. Results

The analysis results were considered in terms of hydraulic and structural behavior. Flow and ground behavior are described in terms of flow vector, pore-water pressure distribution, and ground loading on the lining. Lining performance was investigated in terms of hoop thrusts, bending moments and lining deformation. The influence of local deterioration was highlighted by comparing the results with those from the two extreme hydraulic boundary conditions: fully permeable and impermeable boundaries.

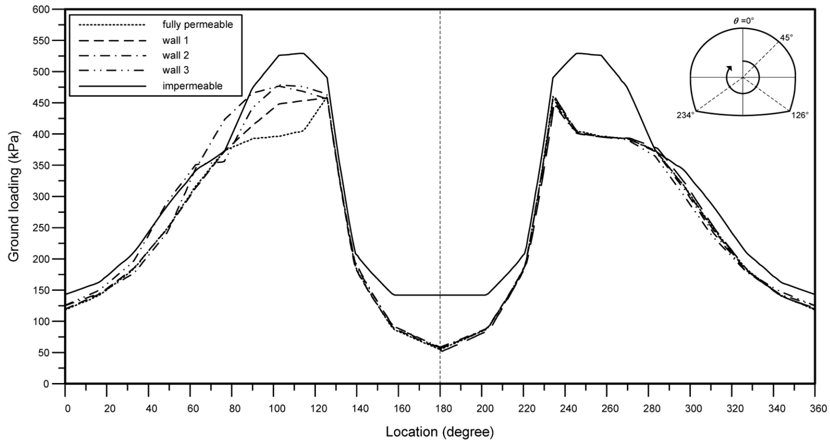
4.1 Flow behavior

Fig. 6(a) presents flow velocity vectors in the ground adjacent to the tunnel. It is shown that there is no significant seepage along the lining of which drainage system is hydraulically deteriorated. Seepage concentration is found at the boundary between deteriorated and non-deteriorated zones. An increase in deteriorated range has increased the magnitude of the vectors at the boundary. It is noteworthy that an increase in seepage velocity may accelerate the movement of soil particles, cause clogging of the drainage system, and consequently extend the deteriorated zones.

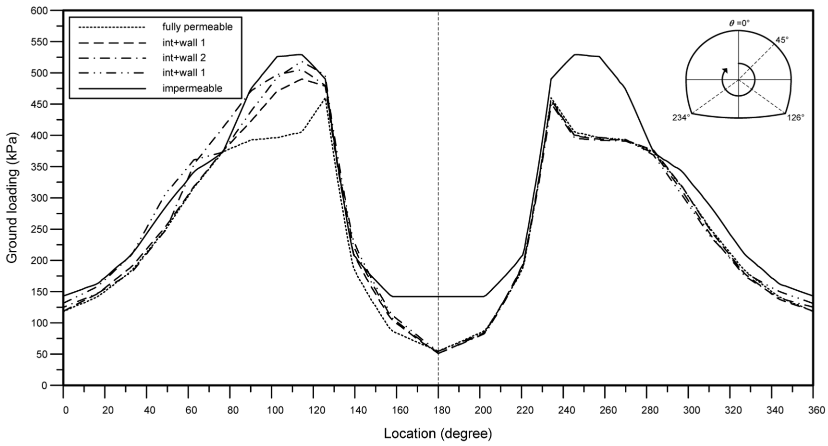
Fig. 6(b) shows the pore-water pressure distributions. In deteriorated zones, pore-water pressure increased considerably. Comparing the profiles, pore-water pressures at the centre of the deteriorated zones approache hydrostatic pore-water pressures with an increase in deteriorated range.

4.2 Ground loading

Frequently the term of earth pressure is used in describing the ground loading. However, earth pressure has been interpreted to mean that soil stresses are distributed directly on the lining to active



(a) Wall deterioration

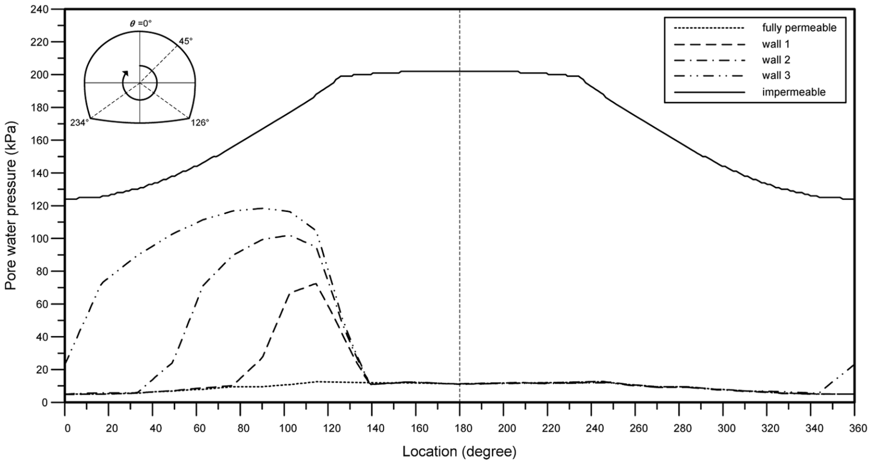


(b) Invert and wall deterioration

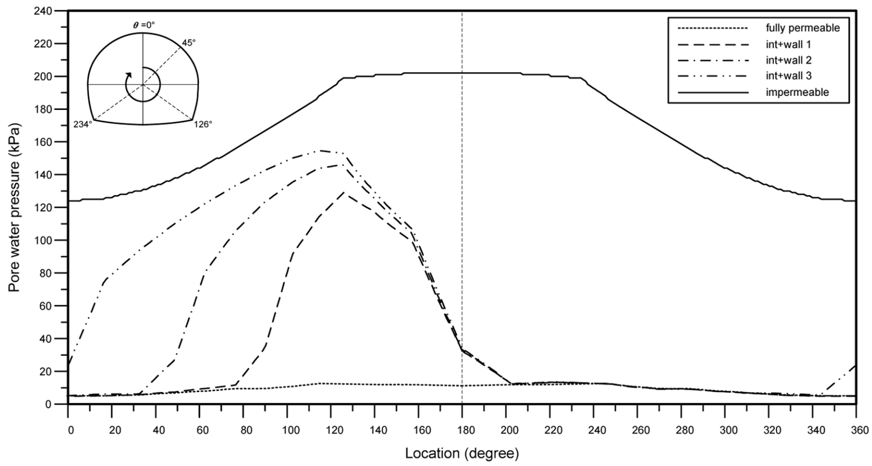
Fig. 7 Ground loadings on the lining

or at-rest conditions. In this paper, the term 'ground loading' is used. Only the normal ground loading on the lining are considered.

Ground loadings are total normal stresses acting on the lining and consisted of effective stress and pore-water pressure components. Fig. 7 shows the distribution of ground loading on the lining. Generally the effects of local deterioration were not significant, and they are in between fully permeable and impermeable cases. However, inspection of these distributions identifies the effect of local hydraulic deterioration of drainage systems. Hindrance of flow into the tunnel increases ground loading over the range where hydraulic deterioration has occurred. In deteriorated zones the ground loadings has increased and approached to those of the impermeable case, meanwhile in non-deteriorated zones they are little changed and almost the same with those of the permeable case. It is interesting to note that for both wall 3 and int+wall 3 Cases slightly higher ground loadings were obtained on a tunnel shoulder than those of the impermeable case.



(a) Wall deterioration

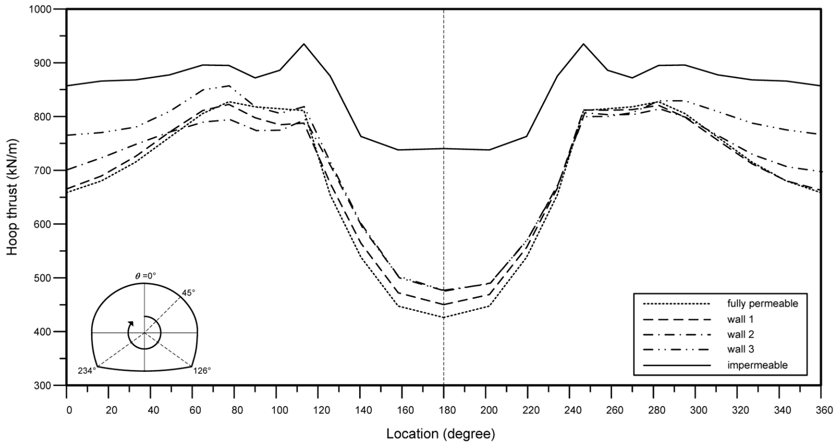


(b) Invert and wall deterioration

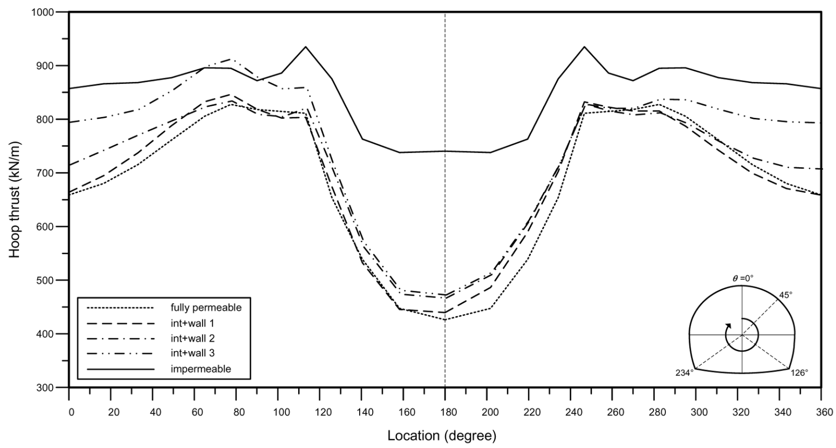
Fig. 8 Pore-water pressure on the lining

Fig. 8 shows pore-water pressure distribution on the linings and compares with the fully permeable and the impermeable cases. Hindrance of flow has increased pore-water pressure noticeably in the deteriorated zones of the lining. In addition pore-water pressure distribution became highly non-symmetric. Maximum pore-water pressure was found at around the centre of the deteriorated zones, and increased with an increase in the range of the deterioration. When the right half of a tunnel drainage system is clogged, the maximum pore-water pressure approaches to about 75% of hydrostatic pressure. An increase in pore-water pressure may cause or accelerate leakage through cracks in the concrete lining.

Ground loadings presented in this paper are total stress. Despite an increase in pore-water pressure changes in ground loadings are not significant as presented in Fig. 7, which means that the proportion of pore-water pressure in the ground loadings has increased, meanwhile almost the same amount of the effective stress in ground loadings has decreased.



(a) Wall deterioration



(b) Invert and wall deterioration

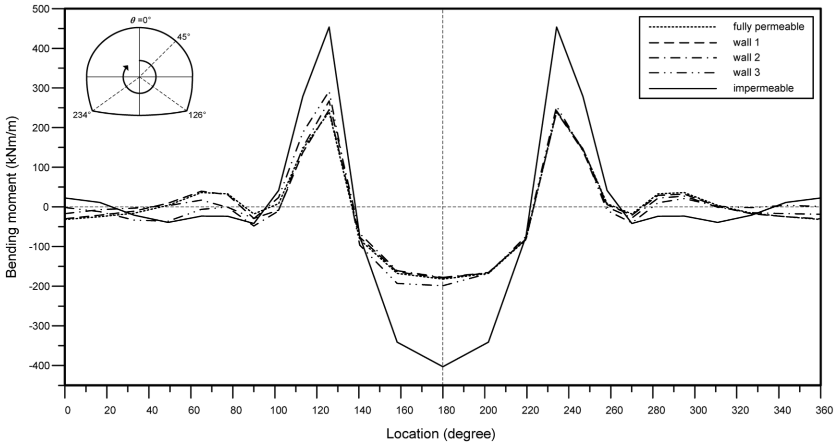
Fig. 9 Hoop thrusts in the linings

4.3 Lining performance

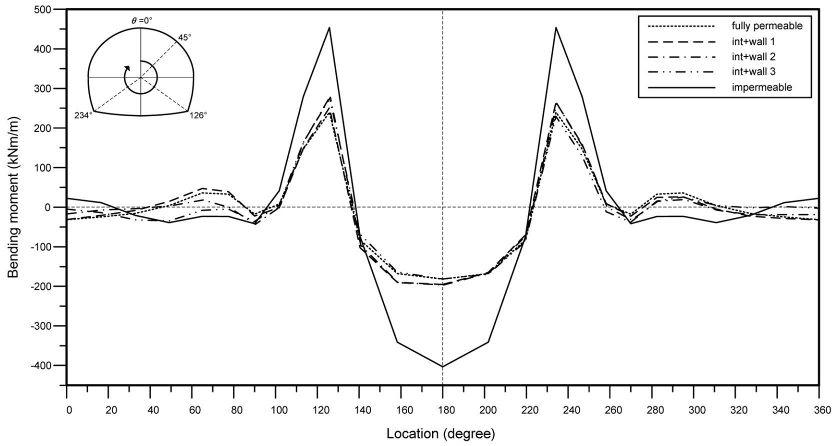
Fig. 9 presents hoop thrusts in the linings. Although deterioration occurs locally, the hoop thrust has changed all over the lining. Generally the hoop thrusts are in between those of the fully permeable and the impermeable linings except the int+wall 3 Case where some hoop thrusts exceed those of the impermeable lining. It is interesting to note that although pore-water pressure increased locally as shown in Fig. 8, the hoop thrusts have increased almost all over the linings.

Fig. 10 presents the bending moments in the linings. The influences of local deterioration are generally small for both the wall and the int+wall Cases. Distribution of bending moments is in between those of the permeable and the impermeable Cases. It shows that all values are very close to those from the fully permeable cases. Although the magnitude of bending moments is not significant, slight asymmetric distribution is obtained and likely to cause asymmetric deformation.

Fig. 11 shows the deformation of tunnel linings. Deterioration of the right side wall moves the upper part of the tunnel lining to the right. An increase in deteriorated range increases the lateral

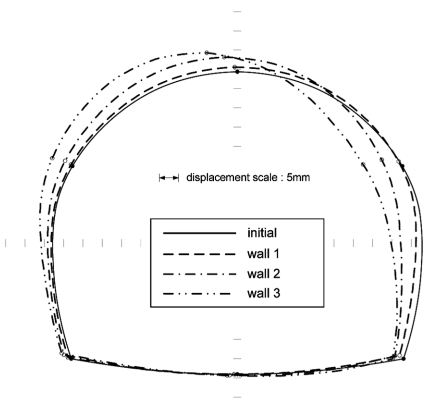


(a) Wall deterioration

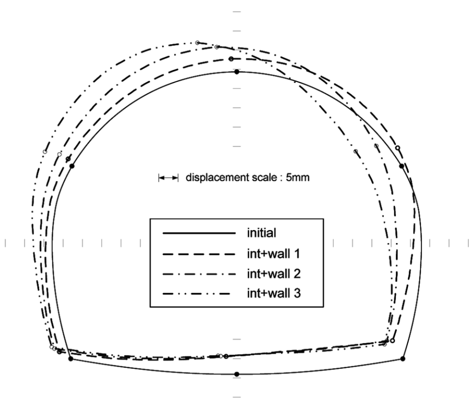


(b) Invert and wall deterioration

Fig. 10 Bending moments in the linings



(a) Wall deterioration



(b) Invert and wall deterioration

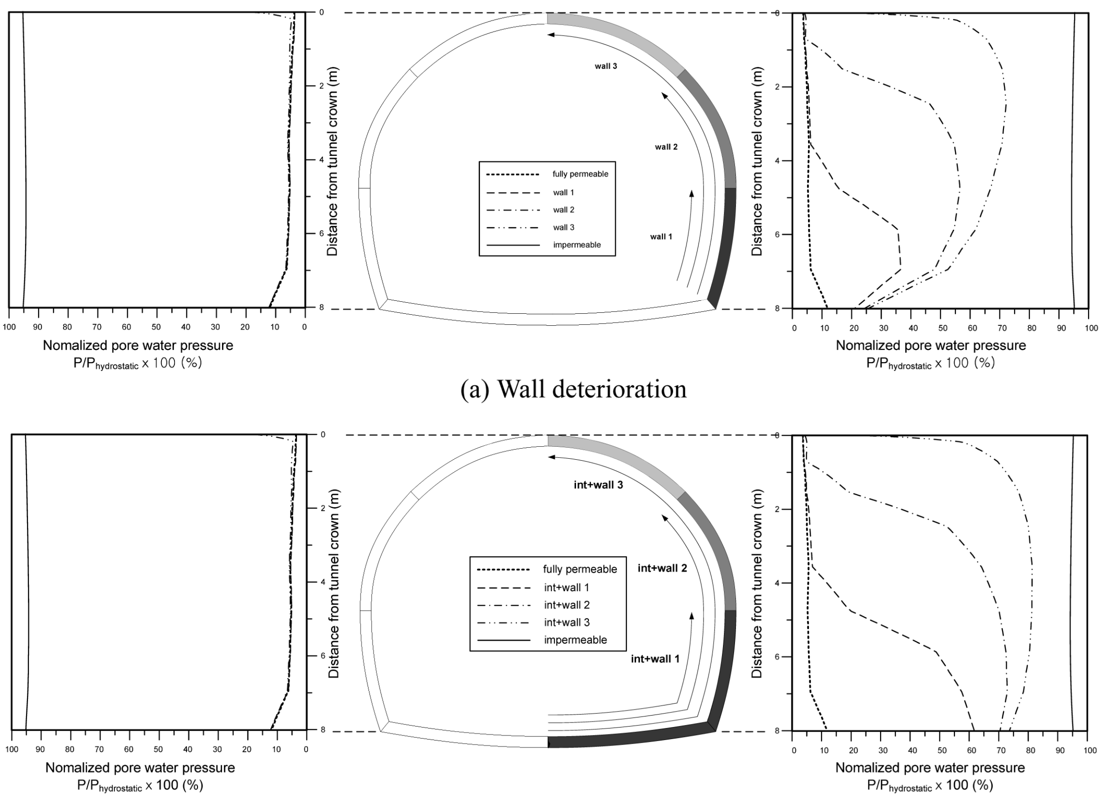
Fig. 11 Deformation of linings

movements. Wall and invert deterioration caused lateral and differential uplift movements. This type of movements indicates that local hydraulic deterioration of a drainage system may cause significant longitudinal (in the direction of tunnel axis) differential deformation at the interface between deteriorated and non-deteriorated zones.

5. Consideration in secondary lining

It has shown that blockage of seepage routes has the effect of reducing the permeability of drain filter. Therefore, additional pore-water pressures will be developed and consequently cause detrimental effects on the tunnel. In this case only the permeability of the primary lining (or single shell lining) is considered. However, in a tunnel with both primary and secondary linings, clogging of a drainage system would occur in a filter layer, and the increased pore-water pressures act on the secondary lining (Poscher and John 1993). Meanwhile the net pore-water pressure on the primary lining in this case would be very small.

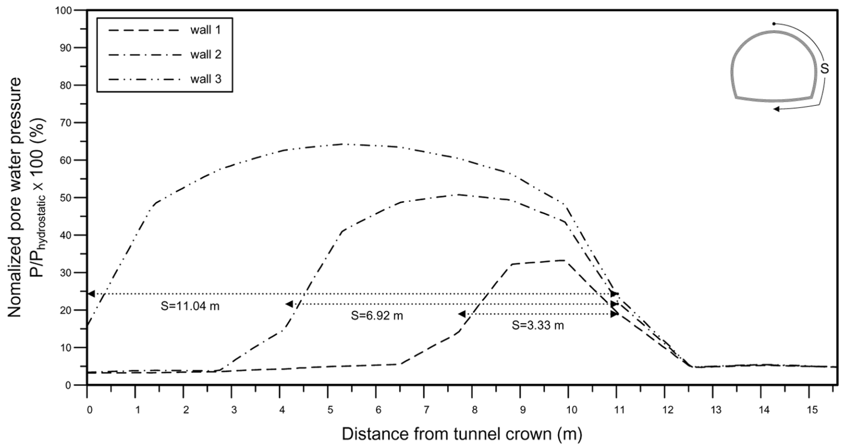
The secondary lining is customarily designed to withstand only a small fraction of the overburden pressure. In particular, in the case where the secondary lining is installed a long time after completion of the primary lining, it is frequently assumed that the primary support system absorbs



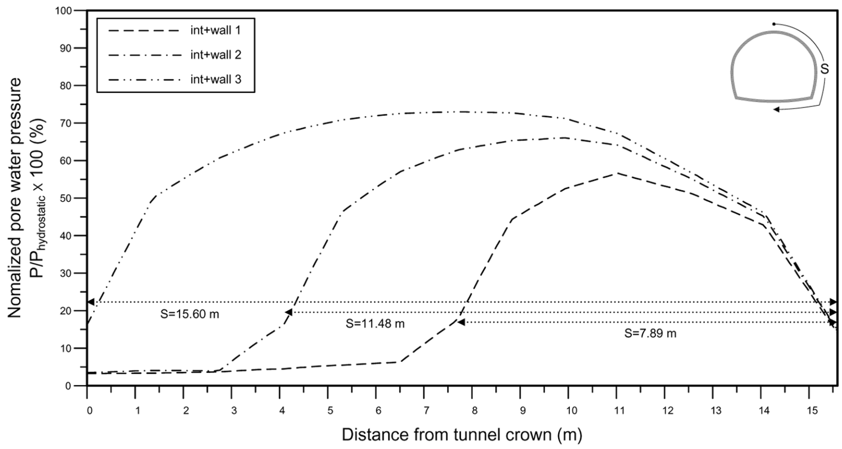
(a) Wall deterioration

(b) Invert and wall deterioration

Fig. 12 Pore-water pressure curves for lining with local deterioration



(a) Wall deterioration



(b) Invert and wall deterioration

Fig. 13 Water load on the secondary lining

or resists all external loads and redistributes unequal pressures before the secondary lining is installed. Therefore, the secondary lining is frequently considered a non-structural member, and is used principally for interior surface treatment. However, as identified in this paper and supported by the failure cases (Ferreira 1995, Lee *et al.* 1996), the most obvious source of loads on the secondary lining is the pore-water pressures. Therefore, it is required to consider pore-water pressure in designing the secondary lining.

If it is assumed that the permeability of the primary lining is the same as that of the surrounding ground and that the permeability of the drainage system is less than that of the soil, then the water pressure presented in Fig. 8 will now act directly on the secondary lining. The magnitude of pore-water pressures acting on the secondary lining depends on the hydraulic capacity of the drainage system. Shin *et al.* (2005) investigated the variation of pore-water pressure loads on a secondary lining of which drainage systems are entirely deteriorated, and highlighted that the relative permeability is the only influencing factor. The distribution of pore-water pressure due to local

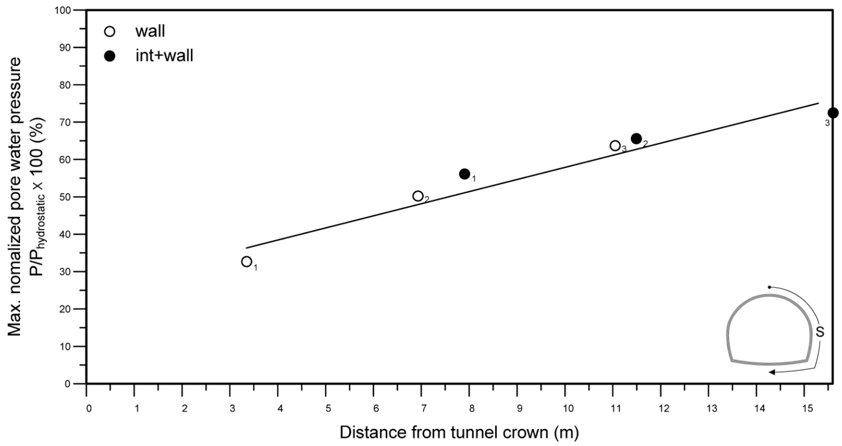


Fig. 14 Maximum pore-water pressure with varying deteriorated ranges

deterioration is, however dependent on the pattern of deterioration as shown in Fig. 12. Moreover, pore-water pressure influenced-range is extended by about 20% of the deteriorated length from the deterioration boundary.

To establish design water load on the secondary lining for local deterioration, further investigation of pore-water pressure development mechanism was made. Fig. 13 shows the profiles of pore-water pressure represented in terms of the length of deteriorated range. The pore-water distribution can be represented by using quadratic function. The consequent design water load on the secondary load can be defined as

$$\frac{p}{p_o} = \frac{1}{0.49s^2}\left(\frac{p}{p_o}\right)_{\max}(X-0.7s)(X+0.7s) \tag{5}$$

where X is the distance (m) from the centre of deterioration range ($0 \leq X \leq 0.7s$), $(p/p_o)_{\max}$ is the pore-water pressure ratio (%) at the centre of deteriorated range, and s is the length (m) of deterioration.

The $(p/p_o)_{\max}$ is found at the centre of deteriorated zone. Fig. 14 shows that the maximum pore-water pressure increases linearly with an increase in deteriorated length. The $(p/p_o)_{\max}$ can be evaluated using the Eq. (6).

$$\left(\frac{p}{p_o}\right)_{\max} = 3.3s + 25 \tag{6}$$

If deterioration range is detected using the geophysical surveys, or design conditions are prescribed, then the design water pressure on the secondary lining can be evaluated using Eqs. (5) and (6).

6. Conclusions

The coupled structural and hydraulic behavior of tunnels due to local deterioration of a drainage system was investigated using the coupled finite element method. The coupled structural and hydraulic behavior of the lining was modeled in two dimensions by using a combination of beam and solid elements. Numerical representation of hydraulic deterioration is simply made by reducing the permeability of the solid lining elements. The significance of the local deterioration at the tunnel lining has been highlighted throughout this paper. Analysis results revealed that

(1) Hydraulic deterioration of a drainage system increases pore water pressure and seepage velocity significantly, which may accelerate the movement of soil particles, cause clogging of the drainage system, and consequently extend deteriorated zones.
(2) The magnitude of the pore-water pressure on a lining is strongly dependent on the deteriorated areas.
(3) Local deterioration of drainage system changes the contribution of pore-water pressure to the ground loading without changing the magnitude of the ground loading.
(4) Deterioration has increased hoop thrusts almost all over the linings, meanwhile bending moments has little changed.
(5) It is noteworthy that the local deterioration of a drainage system has a significant impact on tunnel deformation particularly in the axial direction.
(6) It is recommended that the deterioration of drainage systems should be considered as one of the critical factors in designing the secondary linings which are usually not designed to cope with pore-water pressure load. Simple method evaluating design water loads for local deterioration is proposed for the conditions considered in this study.

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