

Cohesive zone model-based analyses of localized leakage of segmentally lined tunnels

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ABSTRACT This paper presents a novel approach for simulating the localized leakage behavior of segmentally lined tunnels based on a cohesive zone model. The proposed approach not only simulates localized leakage at the lining segment, but also captures the hydromechanically coupled seepage behavior at the segmental joints. It is first verified via a tunnel drainage experiment, which reveals its merits over the existing local hydraulic conductivity method. Subsequently, a parametric study is conducted to investigate the effects of the aperture size, stratum permeability, and spatial distribution of drainage holes on the leakage behavior, stratum seepage field, and leakage-induced mechanical response of the tunnel lining. The proposed approach yields more accurate results than the classical local hydraulic conductivity method. Moreover, it is both computationally efficient and stable. Localized leakage leads to reduced local ground pressure, which further induces outward deformation near the leakage point and slight inward deformation at its diametrically opposite side. A localized stress arch spanning across the leakage point is observed, which manifests as the rotation of the principal stresses in the adjacent area. The seepage field depends on both the number and location of the leakage zones. Pseudostatic seepage zones, in which the seepage rate is significantly lower than that of the adjacent area, appear when multiple seepage zones are considered. Finally, the importance of employing the hydromechanical coupled mechanism at the segment joints is highlighted by cases of shallowly buried tunnels subjected to surface loading and pressure tunnels while considering internal water pressure.

KEYWORDS segmentally lined tunnel, localized leakage, cohesive element, hydraulic behavior, numerical modeling

1 Introduction

Localized leakage through the joints of segmental lining is one of the main reasons for the ground settlement and failure of shield tunnels [1]. Understanding the effect of groundwater infiltration on the responses of linings and the surrounding ground is crucial; however, this is difficult to achieve during the operation of segmentally lined tunnels, which have garnered significant attention in the past decades. Groundwater infiltration in segmentally lined tunnels can be classified into the following three cases.

Case 1: Slight leakage may occur during operation

owing to the hydraulic deterioration of joints, which may result in long-term settlement [2–5], thus jeopardizing operational safety. Wu et al. [5] summarized the leakage problems that occurred in the Shanghai metro and reported the occurrence of continuous groundwater leakage ($> 1 \text{ L} \cdot \text{m}^{-2} \cdot \text{d}^{-1}$ exceeding the permitted amount of $0.2 \text{ L} \cdot \text{m}^{-2} \cdot \text{d}^{-1}$) in many tunnels. Zhang et al. [6] reported that tunnels may be subjected to partial leakage conditions, where the joints at the waist of the lining are the main leakage areas. Wu et al. [4] reported that leakage typically occurs at circumferential, longitudinal, and cross joints, whose probabilities of occurrence contribute 55.26%, 26.32%, and 13.16% to the total leakage areas, respectively.

Case 2: Unexpected water infiltration occurs along with

a significant amount of soil and water gushing. This typically occurs when tunnels are subjected to severe disturbances [7,8] as well as geological hazards induced by fragile aquifers [9,10].

Case 3: A significant amount of leakage occurs during drainage through drainage holes, which are typically adopted in deeply buried tunnels with high water pressure [11–14]; this active drainage method reduces the internal forces of the lining.

The flow rate through the drainage holes is much higher than the joint seepage in Case 1 but more stable and controllable compared with that in Case 2. This study focuses on the Cases 1 and 3.

Several analytical solutions have been proposed to analyze groundwater infiltration [15–19], most of which focused primarily on the groundwater inflow rate and seepage field of the stratum. For instance, a conformal mapping technique has been adopted to obtain an exact solution [15,16], which is more general than the approximate solution of Goodman's formula [20]. Furthermore, utilizing the image tunnel method efficiently simplifies the boundary of the semi-infinite seepage field [17] by transforming it into a superposition of two infinite seepage fields. However, these studies simplified the lining and its local features to simplify

analytical calculations. Therefore, only a few can obtain the actual response of the lining structure or localized leakage behavior subject to groundwater infiltration.

Compared with analytical methods, numerical simulations can solve more general cases of groundwater infiltration by preserving the details to the maximum extent. In addition, transient analysis of coupled seepage and mechanical behavior is applicable in numerical modeling, based on which a dynamic mechanical and hydraulic interaction between the soil and lining can be obtained [21]. Generally, numerical methods for simulating the infiltration groundwater into a tunnel include the hydraulic conductivity method, which considers the spatial variation of hydraulic conductivity, and the inflow rate method, which specifies the water inflow rate. The former is the most typically used method and can be classified into equivalent hydraulic conductivity, partial hydraulic conductivity, and local hydraulic conductivity methods, as shown in Fig. 1.

Figures 1(a) and 1(b) show the equivalent and partial hydraulic conductivity methods, respectively. The permeability of the leakage area can be calibrated via trial calculations based on the seepage rate measured in actual cases [6]. Figures 1(c)–1(e) show several modeling techniques for describing the local leakage behavior; in

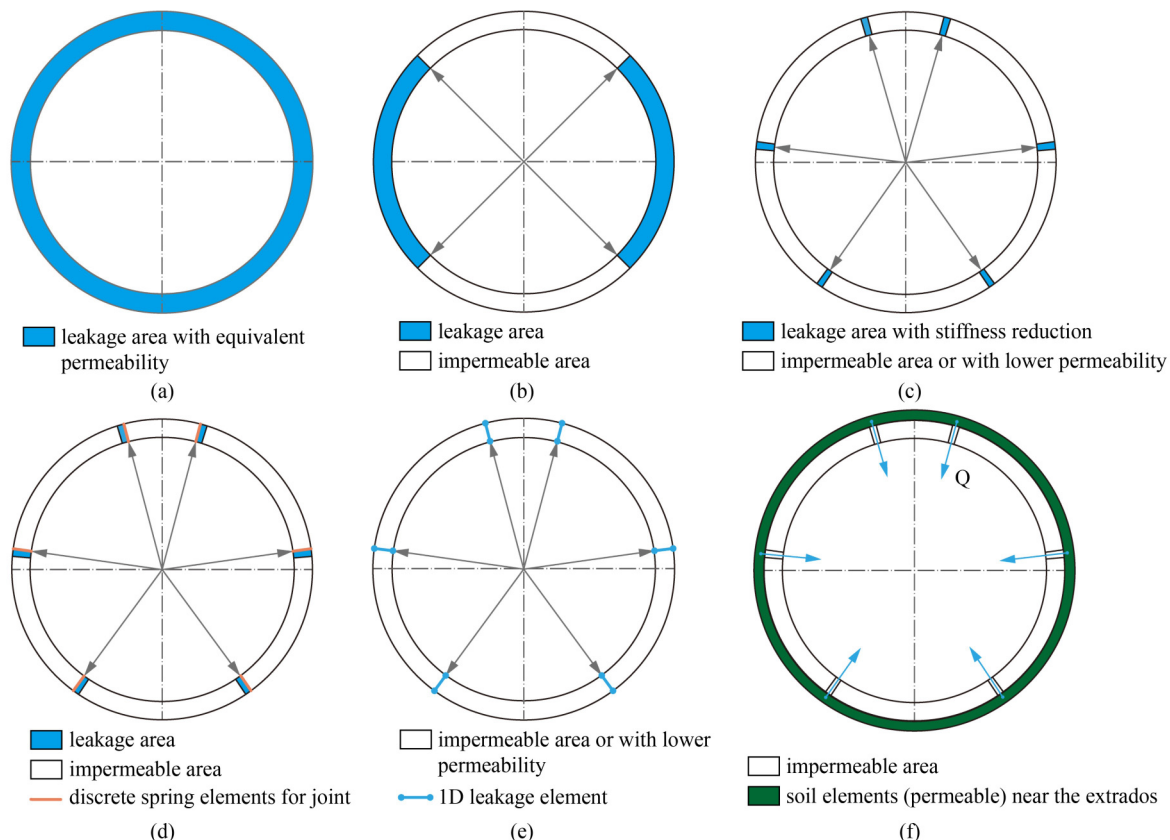


Fig. 1 Existing numerical modeling methods for groundwater infiltration: (a) equivalent hydraulic conductivity method; (b) partial hydraulic conductivity method; (c) local hydraulic conductivity method embedded with localized leakage area; (d) local hydraulic conductivity method embedded with localized leakage area and discrete spring elements; (e) local hydraulic conductivity method embedded with one-dimensional leakage elements; (f) inflow rate method.

these techniques, a highly permeable area distinguished from the intact lining of concrete is specified. Figure 1(f) shows a schematic illustration of the inflow rate method, where the leakage rate is directly provided at a specific location. By implementing stress- and seepage-coupled leakage behaviors [22–24], the leakage rate can either be constant [22] or nonlinear [23,24]. A summary and comparison of these modeling methods are presented in Table 1.

Although existing numerical methods, particularly the local hydraulic conductivity method, are efficient in simulating groundwater infiltration problems, the localized leakage area is simulated as a continuum partitioned subjectively from the lining entity, whereas its permeability coefficient is obtained via trial calculations based on specified seepage rates and cannot differentiate levels of local hydraulic deterioration. If the leakage area is similar to the actual size, then the model may be complicated and requires a long simulation time. Furthermore, as shown in Fig. 2, the leakage behavior of the joint is coupled with its mechanical response. The longitudinal joint of the segmented tunnel lining experiences both axial force and bending moment when subjected to external water-soil pressures, which results in a relative rotation between the adjacent segment faces at the joints. Joint deformation may change the hydraulic aperture, which causes the redistribution of the seepage field. The change in the seepage field results in a change in the external and internal loads exerting on the lining structure, which consequently induces a mechanical response at the joint. This hydraulic-mechanical coupling cycle is related to the response of the entire tunnel lining and, consequently, affects the soil-structure interaction. In other words, the local hydraulic conductivity at the joint should not be constant but should depend on the joint deformation, i.e., the opening or closure of joints will increase or decrease the local hydraulic conductivity,

respectively. Nevertheless, this coupling between hydraulic conductivity and joint deformation is absent in existing approaches. Hence, a new method is required to accurately simulate the effect of local leakage on the behavior of segmentally lined tunnels.

In this study, a cohesive zone model (CZM) combined with an effective aperture was employed to model localized leakage areas in segmented tunnel linings. We used this optimized local hydraulic conductivity method, where zero-thickness cohesive elements were implemented across the lining, to investigate leakage-induced seepage fields, lining responses, and hydraulic-mechanical behaviors at the leakage area.

The remainder of this paper is organized as follows: First, a simplified CZM for simulating local leakage is presented, based on which its two-dimensional (2D) form is derived. The model is verified by comparing its results with results obtained from an existing drainage test, in which drainage holes were simulated based on a constant hydraulic aperture. The merits of the proposed simulation approach over the classical local hydraulic conductivity method are discussed. A parametric study that considers various influential factors is conducted. Finally, the importance of incorporating hydraulic-mechanical behaviors at the segment joints is highlighted.

2 Localized leakage simulation based on CZM

2.1 Cohesive elements with pore fluid flow properties

To simulate the ground movement and response of a segmented tunnel lining under localized leakage, we herein propose an optimized local hydraulic conductivity method based on the CZM. In this method, zero-thickness cohesive elements are implemented in the commercial

Table 1 Summary and comparison of different methods for modeling lining permeability

modeling methods	reference	software	strengths	imperfections
Fig. 1(a): equivalent hydraulic conductivity method	[25]	ICFEP (Imperial College Finite Element Program)	It assumes that the entire tunnel drainage is uniform, with a single parameter representing the lining permeability.	It disregards the difference in the local leakage behavior of joints, handholes, and segments. The lining permeability is difficult to calibrate. The ground movement and pore pressure may be inaccurate when applied to a shallowly buried tunnel [4].
	[26,27]	PLAXIS 2D		
	[28]	ABAQUS		
Fig. 1(b): partial hydraulic conductivity method	[29]	ABAQUS	It considers the variation in leakage behaviors throughout the lining structure based on field survey data of leakage [6].	It is similar to the equivalent hydraulic conductivity method and does not offer a refined simulation of local leakage.
	[6]	ABAQUS		
	[30]	FLAC		
Figs. 1(c)–1(e): local hydraulic conductivity method	[21]	ICFEP	The local leakage behavior can be explicitly simulated.	The permeability coefficient and the width of the leakage area cannot be determined easily. Moreover, the coupling between the hydraulic and mechanical behaviors at the leakage area is not considered.
	[4,31]	ABAQUS		
	[32]	FLAC		
Fig. 1(f): inflow rate method	[22]	not mentioned	The leakage rate is applied directly on the soil near the tunnel extrados; thus, the hydraulic conductivity need not be specified to the lining.	The accuracy of seepage field relies on the prescribed leakage rate or the implemented hydromechanical leakage behavior.
	[23,24]	ABAQUS		

software ABAQUS [33], as schematically illustrated in Fig. 3. Each cohesive element contains three layers of nodes, in which the middle layer of the nodes performs pore fluid flow, whereas the top and bottom node layers discretize the continuous entity, thus adding a traction–separation interaction mechanism at a specified location.

The pore fluid flow was separated into normal and tangential flows along the cohesive elements. The former describes the leak-off between a cohesive element and adjacent entities, whereas the latter describes the flow through the middle channel. Groundwater infiltration was assumed to be caused primarily by tangential flow along the elements. When the gap opens along the cohesive element, the tangential flow is defined as a Poiseuille flow, where an incompressible Newtonian fluid is considered [33].

$$qd = -k_t \nabla p, \quad (1)$$

$$k_t = \frac{d^3}{12\mu}, \quad (2)$$

where q is the flow rate density, d is the aperture of the gap, k_t is the permeability coefficient of the tangential flow, ∇p is the pore-pressure gradient, μ is the fluid viscosity, which was set to 0.001 Pa·s as the viscosity of pure water by default.

The linear elastic traction–separation law for 2D

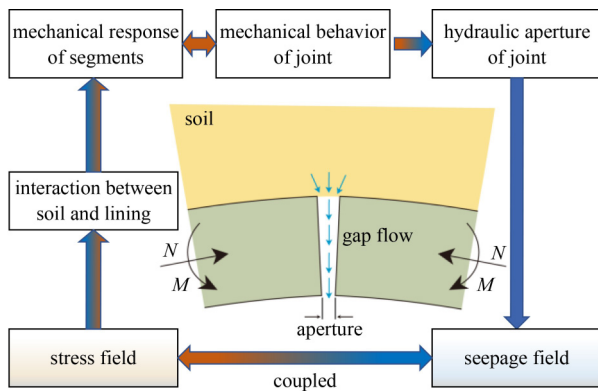


Fig. 2 Schematic illustration of hydraulic and mechanical coupling cycle of gap flow in stratum-structure model.

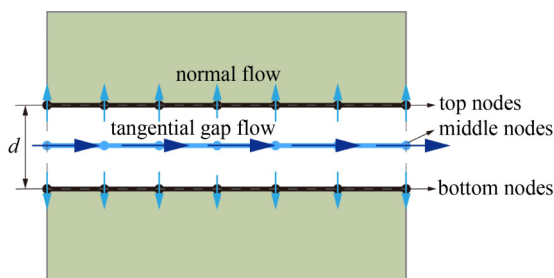


Fig. 3 Cohesive elements with initial aperture.

problems was adopted for the top and bottom node layers of the cohesive elements [33].

$$\mathbf{t} = \begin{Bmatrix} t_n \\ t_s \end{Bmatrix} = \begin{bmatrix} E_{nn} & E_{ns} \\ E_{ns} & E_{ss} \end{bmatrix} \begin{Bmatrix} \varepsilon_n \\ \varepsilon_s \end{Bmatrix} = \mathbf{E} \boldsymbol{\varepsilon}, \quad (3)$$

where $\mathbf{t}$ is the nominal traction stress vector, which comprises two components, i.e., the normal traction (t_n) and shear traction (t_s). The elastic constitutive matrix $\mathbf{E}$ comprises three components, E_{nn} , E_{ns} , and E_{ss} . Meanwhile, ε_n and ε_s are the two components of the nominal strain tensor $\boldsymbol{\varepsilon}$, which are defined as

$$\varepsilon_n = \frac{\delta_n}{T_0}, \quad \varepsilon_s = \frac{\delta_s}{T_0}, \quad (4)$$

where δ_n and δ_s are the normal and shear separations, respectively, and T_0 is the original thickness of the cohesive element.

In this study, only the closed and open states were considered in the analysis by directly defining the initial aperture. In addition, we assumed that the pore pressure from the stratum cannot generate hydraulic fractures in the segments; in other words, we did not consider the fracturing behavior of the cohesive elements.

2.2 Coupled gap flow and normal separation

Unlike most local hydraulic conductivity methods, which cannot correlate the fluid flow behavior to the mechanical behavior of the leakage area, the CZM method can couple the gap flow through the leakage area and the normal separation induced by external loading. Figures 4(a) and 4(b) illustrate the closed state ($d = g_0 = 0$) and the constant aperture ($d = g_0 > 0$), respectively, where g_0 is the initial aperture. Figures 4(c) and 4(d) illustrate the manner by which the external axial loads affect the gap separation, which directly changes the tangential flow behavior. The aperture of the cohesive element is expressed as

$$d = g_0 + \Delta g, \quad (5)$$

where Δg represents the increment of gap separation due to external loads, which is equal to the value of the normal separation δ_n . The aperture may vary along the cohesive elements subjected to bending, as shown in Fig. 4(e), where $d_{\max}$ and $d_{\min}$ are the maximum and minimum opening sizes along the cohesive element, respectively.

2.3 Effective aperture in different scenarios

Unlike classical local hydraulic conductivity methods, the proposed method utilizes Poiseuille flow to derive the hydraulic conductivity (Eq. (2)). Because the localized leakage channel may not be smooth in certain slow-

leakage scenarios, where the aperture is difficult to obtain via physical measurements, we adopted an effective aperture to consider leakage scenarios to the maximum extent. An illustrative example is as shown in Fig. 5, where ∇p was set as 0.5 MPa/m to calculate the leakage rate.

The effective aperture will be extremely small when the rubber seals are intact, and only subtle leakage occurs when Poiseuille flow is applicable, as verified via a seal test [34]. For slow-leakage scenarios represented by Case 1, Shi et al. [35] conducted a sealant performance test and fitted the leakage rate with Poiseuille flow equations. The results indicated that the hydraulic aperture was 0.012 mm when the effective contact stress vanished. Once the segment joints completely deteriorate hydraulically, the

aperture will increase significantly, and the leakage path will be similar to that of a rock fracture [36]. For this situation, Guan et al. [37] indicated that the effective joint aperture may surpass 0.1 mm. For scenarios with a larger leakage rate such as Case 3, the leakage channels (drainage holes) are smooth and macroscopic, and the effective aperture is approximately equal to the mechanical aperture. Accordingly, we classified the effective aperture into three categories: subtle leakage ($d < 0.01$ mm), slow-leakage ($0.01 \text{ mm} < d < 0.1$ mm), and significant leakage ($d > 0.1$ mm). Combined with this concept, the proposed CZM can consider different leakage scenarios with explicit hydraulic conductivities.

3 Model validation

To verify the applicability of the proposed approach for simulating local leakage, a 2D finite element method (FEM) model was established based on a test performed by Yu et al. [13]. The test was performed to analyze the seepage effect of a drainage-segmented lining under high water pressures in the Qinghai-Tibet Plateau area. The drainage holes were simulated using the CZM with a constant hydraulic aperture corresponding to the case shown in Fig. 4(b).

3.1 Test overview

The geometric similarity ratio of the test was set to 40, and the basic similarity ratios of the density and permeability were set to 1. The remaining similarity ratios can be derived based on the three ratios mentioned above via dimensional analysis. Considering the influence region of the seepage field induced by groundwater infiltration, the distance between the tunnel and border must be sufficiently large. Owing to the dimensions of the experimental apparatus, a large geometric similarity ratio was adopted. The modeling box was assembled using a seepage-controlling system, an enclosing steel box, a tunnel lining, and a protective filter. The box measured $4.5 \text{ m} \times 4 \text{ m} \times 0.7 \text{ m}$, which corresponded to $180 \text{ m} \times 160 \text{ m} \times 28 \text{ m}$ in the prototype scale. A seepage control system was established to maintain a fixed water level at the boundary during testing. A protective filter was wrapped around the tunnel to prevent blockage of the drainage holes due to the gushing of similar materials.

The enclosed steel box was filled with similar materials mixed with sand and gypsum at a certain ratio. By adjusting the gypsum dosage, three types of similar soils with high permeability ($k_s = 1 \times 10^{-5} \text{ m/s}$), medial permeability ($k_s = 1.15 \times 10^{-6} \text{ m/s}$), and low permeability ($k_s = 1 \times 10^{-7} \text{ m/s}$) were prepared. Three different initial water heads were considered over the tunnel lining during

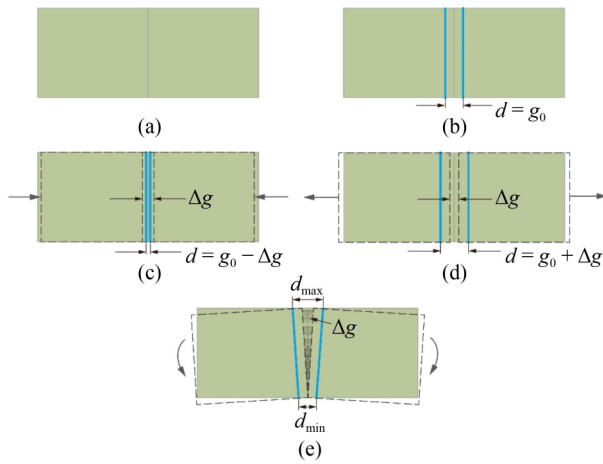


Fig. 4 Change in aperture due to external loads: (a) closed state; (b) opening state with constant aperture; (c) opening state under uniaxial compression; (d) opening state under uniaxial tension; (e) opening state subject to bending (g_0 represents initial aperture, Δg represents the increment of gap separation due to external loads, $d_{\max}$ and $d_{\min}$ are the maximum and minimum opening sizes along the cohesive element, respectively).

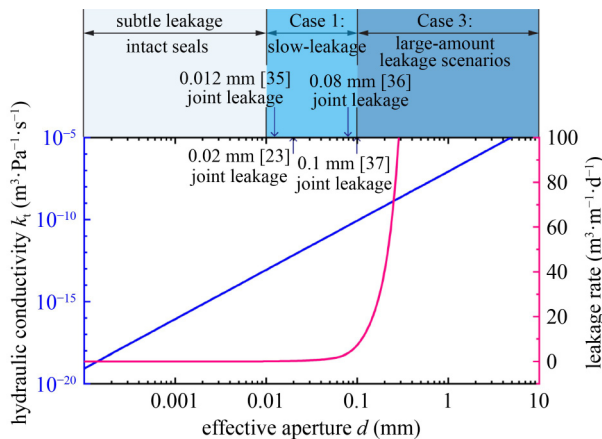


Fig. 5 Correlations among effective aperture, hydraulic conductivity, and leakage rate in different scenarios (∇p was set at 0.5 MPa/m for illustration).

the tests. Only the case with a 90 m water head was simulated as it afforded sufficient details for model validation. The prototype and inner diameters of the tunnel were 8.8 and 8.1 m, respectively, and the diameter of the drainage hole was 5 cm. The spacing between the drainage holes in the longitudinal direction was 1.8 m. The central position of the tunnel was 0.6 m above the box bottom. Similar holes were drilled along the longitudinal and circumferential directions to simulate the leakage behavior of the drainage segments.

3.2 FEM modeling

To demonstrate the advantages of the newly applied CZM, we performed simulations based on a typically used local hydraulic conductivity method, which corresponds to the model shown in Fig. 1(c). The other modeling parameters were identical to those of CZM. Yu et al. [13] converted their model test results to prototype values, and their numerical analyses were based on the prototype model. Therefore, we conducted the current simulations at the prototype scale to perform more comprehensive comparisons. Similar practice was reported in previous studies [38,39].

3.2.1 Model parameters

A 2D numerical model fully coupled with Biot's consolidation theory was established to simulate the seepage field and leakage amount under a high water pressure, where the analysis mode of transient consolidation was employed. The two aforementioned

methods were implemented to model drainage holes with a specific opening amount. Therefore, in this case, the relative deformation of the cohesive elements was not considered, similar to the case where modulus reduction was not considered in the local hydraulic conductivity method.

Figure 6 shows the meshes of the two models, which contain 18344 elements and 18882 nodes, respectively. The mesh size in the vicinity of the tunnel was reduced significantly to insert cohesive elements smoothly and obtain a detailed lining response. The lining was segmented into six elements in the thickness direction to satisfy the numerical analysis requirements of concrete structures [40]. The element type of the stratum and lining was CPE4P, which is a four-node plane stress element coupled with the pore fluid stress. The element type of the cohesive elements was six-node COH2D4P. The model range was consistent with the prototype, i.e., the size of the test box multiplied by the similarity ratio, which was $180\text{ m} \times 160\text{ m}$. The position of the tunnel center was $(0, -136\text{ m})$.

The parameters of the lining and stratum were derived from the prototype parameters listed in Table 2. The lining segment was idealized using elastic and isotropic solid elements. The stratum was idealized as an elastoplastic material based on the Mohr–Coulomb criterion with a friction angle of 30° and cohesion of 0.3 MPa, in accordance with the derived scaling law. Based on the model tests, the permeability coefficients of soil k_s were set to 1×10^{-5} , 1.15×10^{-6} , and $1 \times 10^{-7}\text{ m/s}$, respectively. The permeability coefficient of the concrete lining was set to $1 \times 10^{-10}\text{ m/s}$ according to Neville [41].

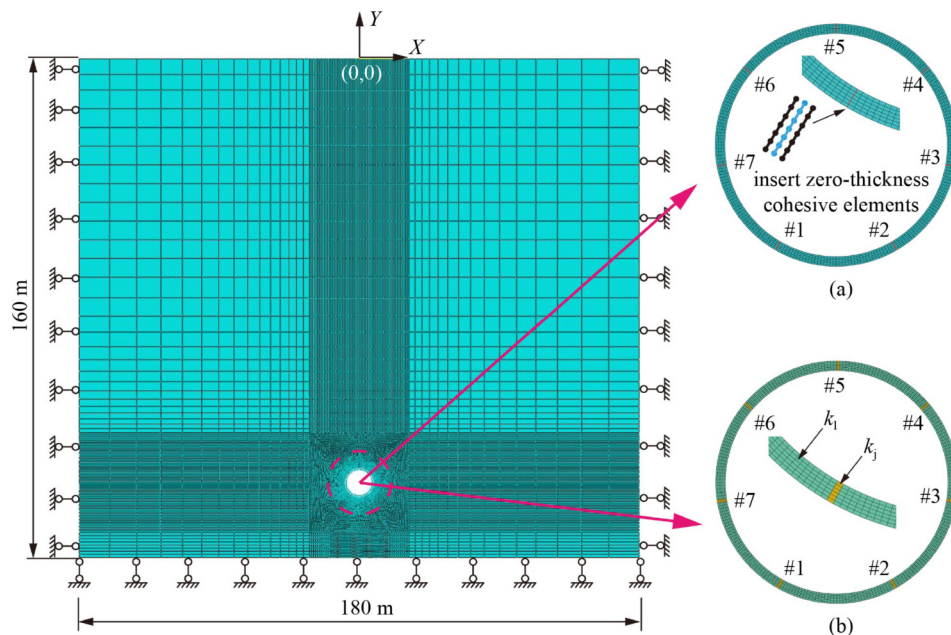


Fig. 6 Meshes for the 2D numerical models: (a) lining model derived from CZM; (b) lining model partitioned via local hydraulic conductivity method (where k_l and k_j refer to the permeability coefficient of the lining and leakage area, respectively).

Table 2 Input physical and mechanical parameters of stratum and lining [13]

material	density (kg/m ³)	elasticity modulus (GPa)	Poisson's ratio	void ratio
stratum	2000	1.5	0.35	0.2
lining	2500	35.0	0.20	0.1

Because the change in seepage rate and stratum seepage field close to the concrete lining was insignificant compared with that of the drainage holes, we only considered the local leakage areas when calculating the seepage rate of water infiltration, which in fact facilitated the comparison of the two modeling methods.

3.2.2 Modeling of local hydraulic behavior

Figures 6(a) and 6(b) show the modeling techniques for the local leakage area using the two different methods. Seven evenly distributed drainage holes were observed over the tunnel cross-section. The drainage holes in the tunnel lining were simplified using a 2D numerical model. The hydraulic gradients along the leakage area were assumed to be consistent in both physical and numerical models, based on which the hydraulic parameters of the two numerical models were derived.

In the CZM, the initial aperture g_0 was derived from the radius (r) and longitudinal spacing (w) of the drainage holes by converting the Hagen–Poiseuille flow to a Poiseuille flow based on the same hydraulic conductivity:

$$d = g_0 = \sqrt{\frac{3}{2} \times \frac{\pi r^4}{w}} = 0.01 \text{ m}. \quad (6)$$

To model the drainage holes using the local hydraulic conductivity method, as shown in Fig. 6(b), the permeability of the materials at the location of these holes was enhanced, whereas the elastic modulus was maintained. The permeability coefficient of the drainage holes, k_{j0} , in the physical model should be infinitely large, because the drainage holes are not waterproof. However, setting such a value in the local hydraulic conductivity method in numerical modeling may result in calculation errors and miscalculations of the seepage field. Therefore, we set k_{j0} to 1 m/s, which is sufficiently large for drainage holes [38]. Subsequently, the k_j of the localized leakage area can be derived as follows: $k_j = k_{j0} A_{j0} / A_j = 1 \times 0.025^2 \pi / (0.1 \times 1.8) = 0.01 \text{ m/s}$, where A_{j0} and A_j are the cross-sectional areas of the drainage hole and the corresponding cross-sectional area of the localized leakage region in the numerical model, respectively.

3.2.3 Boundary conditions

The water level was fixed at $Y = -40 \text{ m}$ based on an unlimited supply of groundwater. The boundary conditions of the model remained the same throughout

the analysis. No horizontal and vertical displacements were imposed along the vertical and horizontal mesh boundaries, respectively. Meanwhile, the intrados of the tunnel and the top mesh boundary were allowed to deform.

As the focus of this study was on leakage-induced long-term behaviors, the variation in excess pore pressure caused by tunneling was disregarded [4]. The hydrostatic field was generated from the beginning and remained constant throughout the analysis. During the internal drainage, the pore-pressure head of the intrados of the tunnel was set to zero to activate infiltration. The results shown below were derived from the final analysis step when the seepage field stabilized. Furthermore, to ensure a continuous hydraulic field, the external nodes of the tunnel and adjacent stratum nodes were merged at the interface, with the assumption of zero relative displacement between them.

3.3 Result analysis

Figure 7 shows a comparison of the internal seepage rates obtained using different methods with those obtained experimentally [13], where different stratum permeabilities were considered. The test results [13] indicate that the seepage rate increased with the permeability. Both the CZM and local hydraulic conductivity method yielded results that were generally consistent with the test results. The results derived from the CZM were more accurate than those obtained from the local hydraulic conductivity method, particularly for the cases with larger k_s . The simulation results obtained from the current study using the local hydraulic conductivity method were similar to the numerical modeling results of Yu et al. [13], which were obtained using the local hydraulic conductivity method.

The seepage rate can be derived using the image method as follows [42]:

$$q_0 = 2\pi k_s \frac{h_s}{\ln(2H_0/b)}, \quad (7)$$

where q_0 is the seepage rate of the tunnel per unit length, H_0 is the hydraulic head between the groundwater level and tunnel center, and h_s is the hydraulic drawdown through the stratum.

$$h_s = \frac{H_0 C (k'_1/k_s)}{1 + C (k'_1/k_s)}, \quad (8)$$

where $C = \ln(2H_0/b) / \ln(b/a)$, where a and b refer to the inner and outer radii of the tunnel, respectively; k'_1 denotes the equivalent permeability of the lining, where the effect of drainage gaps is distributed evenly in the lining [42]. The analytical results derived from the image method are shown in Fig. 7, which are consistent with the test data and numerical results.

A sufficiently long stepping time was adopted such that the seepage field would stabilize eventually. Figure 8 illustrates the time history curves of the internal seepage rate of the CZM and the local hydraulic conductivity method under three different stratum permeability values. The iterative balance of the two methods differed significantly. The tunnel seepage rate derived from the CZM was stable after the first calculation step, unlike that derived using local hydraulic conductivity method, which required a certain amount of time to stabilize. Moreover, as the permeability decreased, the seepage rate required more time to stabilize. If we define the steady-state as the instant when the seepage rate reaches 99.9% of the final value, then the calculation times required to reach the steady-state are 2549 s (0.03 d), 2.29×10^4 s (0.26 d), and 2.96×10^5 s (3.43 d) for $k_s = 1 \times 10^{-5}$, 1.15×10^{-6} , and 1×10^{-7} m/s, respectively. This is because the hydraulic conductivity of the localized leakage area in this method was reduced to accommodate the actual cross-section of the drainage holes. In addition, the results of parametric studies (not shown for conciseness) indicated that the seepage rate increased more rapidly at the beginning and

converged to a larger value when using the local hydraulic conductivity method, which was due to the enlargement of the localized leakage area (k_j was reduced accordingly to maintain an identical seepage velocity). This indicates that subjectively defining localized leakage areas when using this method may yield inaccurate results. Hence, using the CZM is more advantageous as it does not model the local hydraulic area as an entity of porous media.

A closed-up view of the seepage fields near the tunnel in the numerical model is shown in Fig. 9. In the CZM model, as the permeability of the stratum increased, the range of hydraulic connection area between drainage holes decreased (Fig. 9(a)). This is reasonable because a higher stratum permeability accelerates the seepage rate, which decreases the influence area of the single drainage holes.

By contrast, the seepage field barely changed as the stratum permeability varied in the local hydraulic conductivity model, as shown in Fig. 9(b), owing to the relatively large, highly permeable area. Hence, the localized hydraulic response of the adjacent stratum may not be accurate compared with the results of the CZM, even under similar leakage amounts. Partitioning a much smaller localized leakage area may result in a more reasonable seepage field adjacent to the tunnel. However, the calculation efficiency will decrease accordingly. Figure 9 shows that the CZM can be adapted for modeling detailed seepage behavior caused by tunnel drainage.

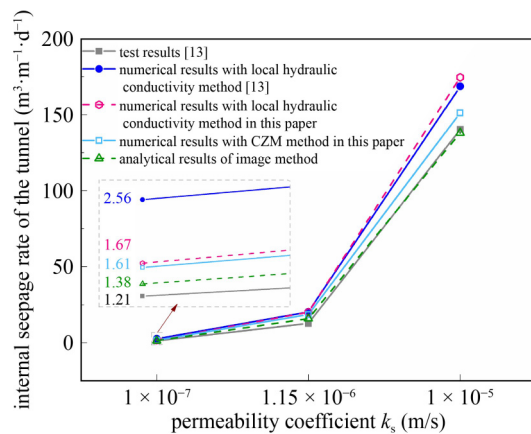


Fig. 7 Seepage rate of water infiltration (values have been converted to prototype values).

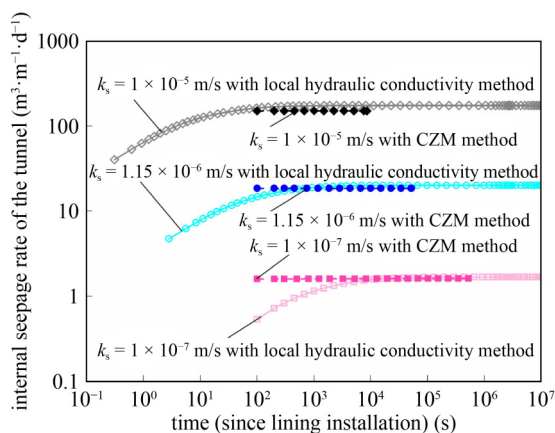


Fig. 8 Time history of localized seepage stabilization.

4 Parametric study

The seepage phenomenon may be affected by many factors. A parametric study was conducted based on the previously verified numerical model, where the same tunnel size, buried depth, and stratum parameters were used. First, a model considering only a single leakage gap at #1, whose location is shown in Fig. 6, was established, based on which the effects of several important parameters on seepage are discussed. Subsequently, the number and spatial distribution of leakage gaps were considered.

4.1 Aperture size

The aperture size dominated the seepage behavior of the leakage area in the CZM. In this study, numerical models with five aperture sizes were established, where $g_0 = 0.01$ and 0.05 mm correspond to the slow-leakage scenario of Case 1 based on the effective aperture derived from test data [35,36], whereas the remaining cases ($g_0 > 0.1$ mm) belongs to Case 3, which reflects significant leakage. Furthermore, a contrasting case without a gap was

established for comparison. In all models, the stratum permeability coefficient k_s was fixed at 1.15×10^{-6} m/s.

Figure 10 shows the stratum response at the external periphery of the lining. Because the tunnel was deeply buried in an aquifer, it sustained a high water pressure, as shown in Fig. 10(a). When $g_0 = 0$ mm, the water pressure was distributed uniformly around the tunnel, where the pore pressures were 920 kPa and 1 MPa at the crown and invert, respectively. As the aperture size increased, the pore pressure near the drainage opening decreased significantly, whereas the water pressure at the other positions decreased by a certain amount, depending on the distance to the leakage gap. When the initial aperture

was approximately 0.5 mm, the pore pressure at the external lining periphery decreased to approximately 0 kPa. In addition, the pore-pressure distribution remained almost unchanged as the opening continued to increase. The effect of seepage on the pore-pressure distribution was insignificant when the initial aperture was 0.01 mm compared to the non-open case.

The localized leakage increased the seepage velocity of the stratum, as shown in Fig. 10(b). The seepage velocities of the non-open case were 2.5×10^{-8} and 3.1×10^{-8} m/s at the crown and invert, respectively. The slight spatial variation in seepage velocity is attributable to the marginal permeability of the lining concrete and the

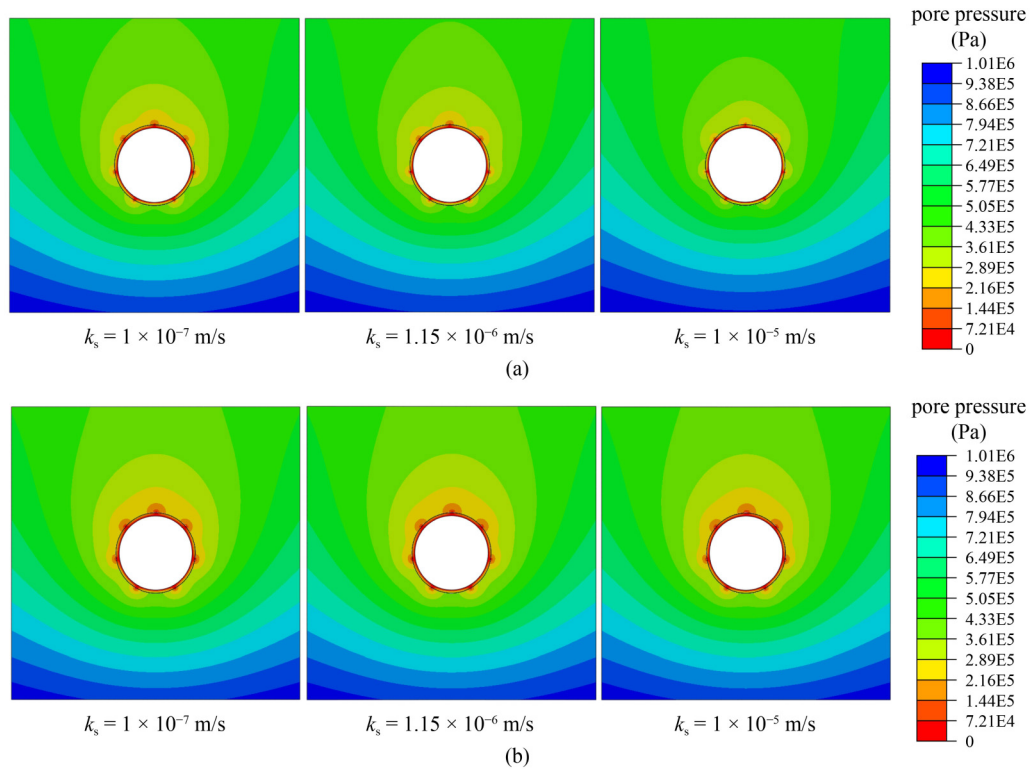


Fig. 9 Seepage fields near tunnel varying with permeability of stratum k_s : (a) numerical results using CZM method; (b) numerical results using local hydraulic conductivity method.

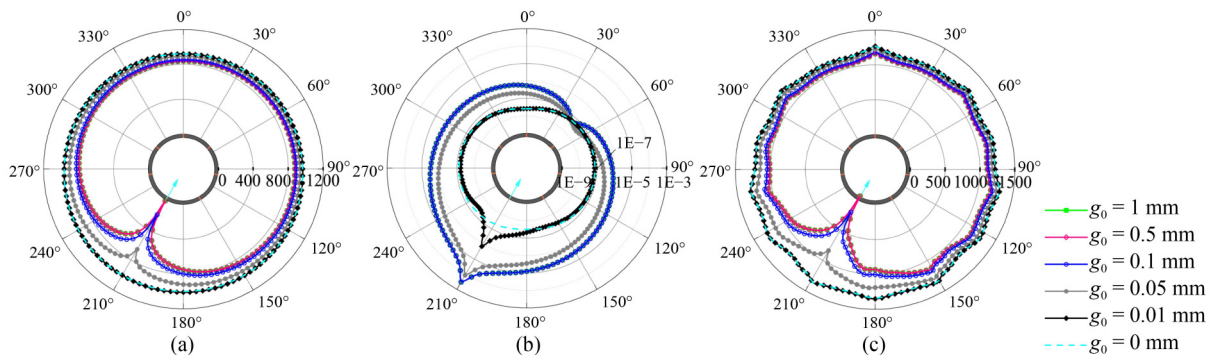


Fig. 10 Hydraulic behavior and mechanical response of stratum at external periphery of lining: (a) pore pressure (kPa); (b) seepage velocity (m/s); (c) ground loading (kPa).

increase in the external water head along the depth. As the initial aperture increased, the seepage velocity increased rapidly, particularly near the leakage gap. Similar to the pore-pressure result, the seepage velocity remained almost unchanged when the opening reached the threshold value, which refers to $g_0 = 0.1$ mm in the current case. However, the seepage velocity did not increase throughout the entire cross-section. In the area diametrically opposite to the leakage point, where the distance to the gap was the farthest, the seepage velocity reduced. This is because the seepage components flowing to the left and right sides are generally the same in local coordinates.

The ground loading comprised the pore pressure and the effective stress of the stratum. The radial distribution of ground loading exerting on the lining is shown in Fig. 10(c). Similar to the pore-pressure distribution, the ground loading decreased at the leakage gap. It decreased to approximately 0 when g_0 was 0.5 and 1 mm, which indicates no ground loading near the localized leakage area. However, a slightly negative value representing subtle tension occurred in these two cases owing to the adoption of the tie interaction mechanism between the lining and stratum.

The change in ground loading induced by seepage resulted in a change in the internal forces of the lining. Figure 11 shows the additional internal forces and deformation along the circumferential direction under different g_0 values for drainage hole #1. Based on Figs. 12(a) and 12(b), the additional bending moment increased, whereas the additional axial force decreased near the drainage location as g_0 increased. The internal forces varied more significantly as g_0 increased. The effect of g_0 became insignificant when it exceeded 0.05 mm. The lining deformed outward near the drainage area (see Fig. 11(c)), whereas it deformed slightly inward in the area diametrically opposite to the drainage area, thus forming an inclined ovalization shape. In other words, the lining elongated diametrically at the leakage gap. The maximum ovalization deformation at a single location was 2.17 mm. Figure 11 shows that seepage may

induce additional deformation of the lining near the leakage gap area owing to the release of ground confinement, which is consistent with the results presented in Fig. 10. Notably, the results of the leakage-induced response might be more unfavorable than those of an actual three-dimensional problem because the localized area was assumed to be infinite in the longitudinal direction.

4.2 Permeability of stratum

In addition to the aperture size, the permeability of the surrounding stratum significantly affects the leakage behavior, as explained in Subsubsection 3.3.1. However, the effect of the leakage mechanism on the single leakage gap remains unclear. As shown in Fig. 12, the seepage rate of #1 increased as k_s increased, but the growth slope varied with the aperture size. When the aperture was 0.01 mm, the seepage rate barely increased as k_s increased, indicating that the aperture size limited the gap flow. When the aperture was greater than or equal to 0.5 mm, the curves of the seepage rate were almost similar to those shown in Fig. 12. At this stage, instead of the aperture, the permeability of the stratum dominated the seepage behavior. However, the effect of the aperture became more significant as k_s increased.

Figure 13 shows the stratum response at the lining extrados vs. k_s , where an aperture of 0.1 mm was considered at #1. Based on Fig. 13(a), the pore pressure decreased significantly with k_s , particularly at locations near #1. This can be explained by the effect of the relative permeability k_l/k_s [21]. When its value equals one, the pore pressure behind the lining approaches zero. The increase in k_s accelerates the seepage velocity of the stratum, as shown in Fig. 13(b). A larger k_s implies more flow inside the stratum and thus more water accumulating near the lining, which results in a higher pore pressure. Additionally, the ground loading decreased with k_s , as shown in Fig. 13(c), which is primarily caused by the decrease in pore pressure.

The seepage through the leakage gap affects the stress

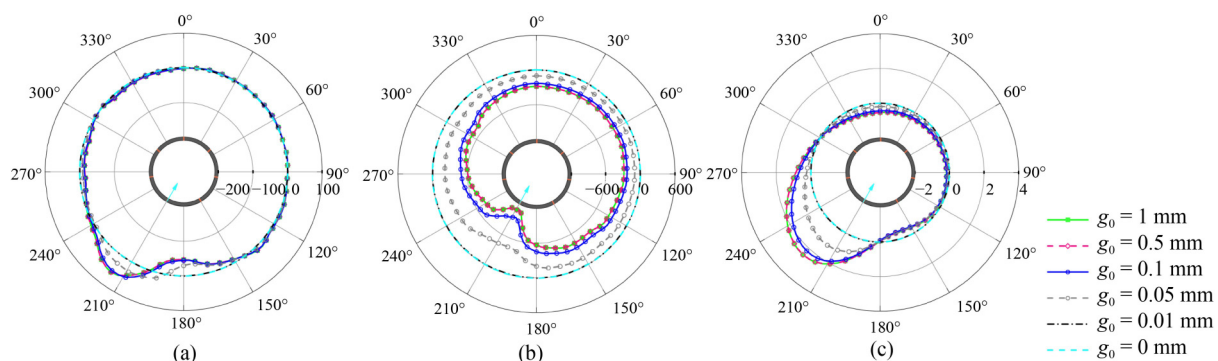


Fig. 11 Mechanical response of tunnel lining: (a) additional bending moments (kN·m); (b) additional axial force (kN); (c) additional radial displacements (mm).

distribution in the stratum near the tunnel. Soil arching occurred around the tunnel after excavation was performed. The centerline of the original stress arch deviated slightly from the vertical direction (see Fig. 14) owing to the disturbance of seepage in the original stress field (Figs. 14(a) and 14(b)). In addition, the increase in aperture as well as the decrease in the stratum permeability facilitated the deviation of the stress arch. To illustrate this phenomenon more clearly, the distribution of the minimum stress vector around the tunnel is presented in Figs. 14(c) and 14(d), which show that the circumferential compressive stress increased near the leakage area. As shown in the detailed contour in Fig. 14(d), local soil arching formed near the drainage hole, with the centerline perpendicular to the lining surface passing through the drainage hole. The deviation in stress arching can be quantified based on the angle of the principal stress with respect to the vertical direction along the survey line marked in Fig. 14(c). For normal soil arching, $\theta_p = 90^\circ$. However, as shown in Fig. 14(e), the minor principal stress deviated from that of the non-gap case in the region where the distance to the tunnel lining was less than 1.2 m. The closer the measurement point to the lining, the larger was the deviation angle. The shear stress (τ_{xy}) over the horizontal and vertical planes was zero when the aperture was zero, which confirmed that

the horizontal and vertical planes were the principal planes. However, as the aperture increased, τ_{xy} became nonzero and varied along the survey line, indicating that the horizontal and vertical planes were no longer the principal planes. Consistent with the rotation in the principal stress direction, τ_{xy} became increasingly smaller in the region where the distance to the tunnel lining exceeded 2.2 m.

4.3 Distribution of leakage gaps

In actual situations, multiple localized leakage areas may be present. The influence zones of different leakage areas may propagate and finally merge during seepage. Eight combinations (#1, #1 + #2, #1 + #6, #1 + #7, #1 + #2 + #7, #1 + #3 + #6, #1 + #2 + #3 + #6 + #7, and #1–#7) of leakage gaps are considered herein based on the model presented in Section 3. The stratum permeability coefficient k_s was set to be 1.15×10^{-6} m/s, and the aperture of the leakage gaps was set to 0.05 mm. Figure 15 shows the seepage velocity (in vector form) around the tunnel for the different cases. The range of the color legend was set as (5×10^{-7}) – (1×10^{-4}) m/s for a better illustration. The results show that the influence area increased with the number of leakage gaps. The seepage fields around adjacent gaps coalesced, whereas an arching “pseudostatic area” emerged between adjacent gaps, wherein the seepage velocity was negligible. The pseudostatic area became more evident as the distance between adjacent gaps increased. Figure 16 shows a detailed vector graph of the pseudostatic area for case #1 + #3 + #6, where the dotted lines partition the stratum into three seepage regions based on the groundwater supply of each leakage gap. Marginal seepage was observed in the pseudostatic area. The pseudostatic area was formed owing to the counterbalance between the far-field water flow in opposite directions to the adjacent gaps. The farther the distance between the adjacent gaps, the higher was the boundary of the pseudostatic area above the line connecting the gaps.

Figure 17 presents the seepage rate of each leakage gap

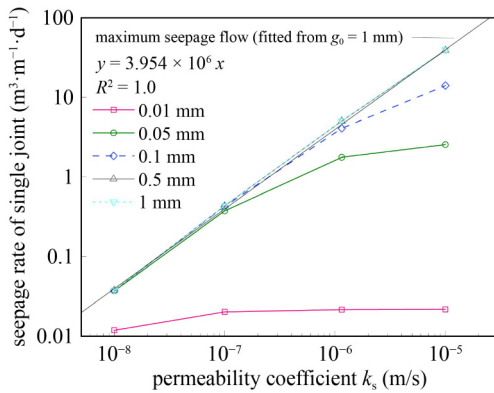


Fig. 12 Effect on seepage rate of single leakage gap.

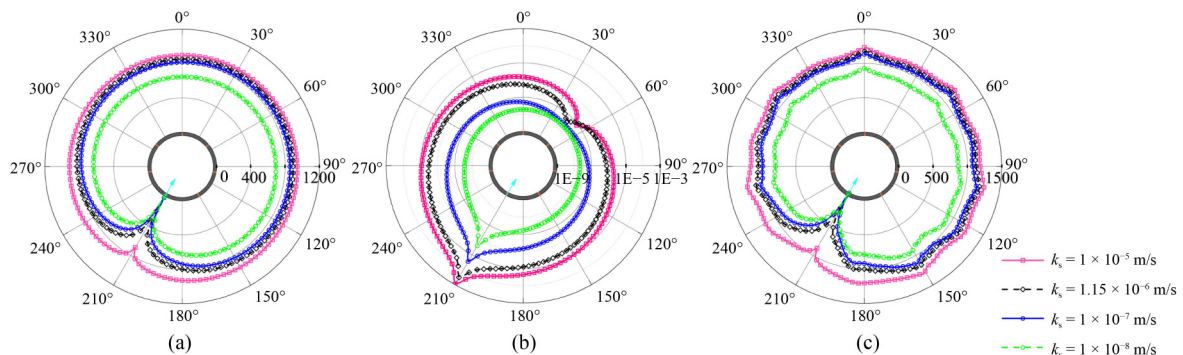


Fig. 13 Effect of stratum permeability on hydraulic behavior and mechanical response of stratum at external periphery of lining: (a) pore pressure (kPa); (b) seepage velocity (m/s); (c) ground loading (kPa).

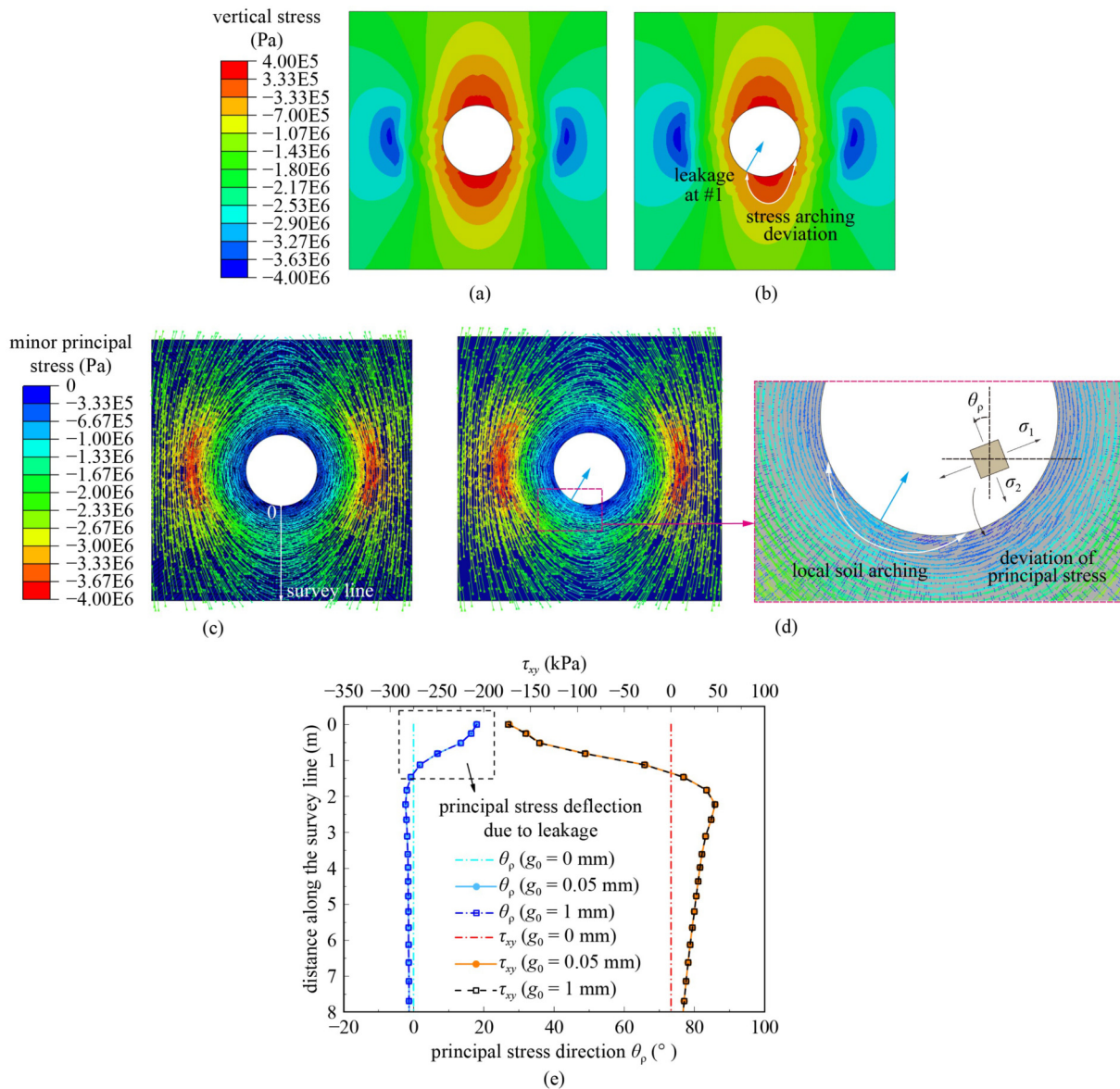


Fig. 14 Mechanical response (effective stress) of stratum induced by leakage when $k_s = 1 \times 10^{-8}$ m/s: (a) vertical stress distribution of non-gap case; (b) vertical stress distribution with $g_0 = 0.05$ mm at #1; (c) minor principal stress distribution of non-gap case; (d) minor principal stress distribution when $g_0 = 0.05$ mm at #1; (e) deviation of principal stress along survey line.

under different combinations of gap locations. For all gaps, the seepage rate decreased as the number of leakage gaps increased. The maximum seepage rates occurred consistently at gaps #1 and #2, where the hydraulic head is the largest, whereas the minimum seepage rate was recorded at #5, where the hydraulic head is the smallest along the tunnel periphery. However, instead of being equal (case #1 + #2), the seepage rate at #1 was lower than that at #2 in the opening case of #1 + #2 + #7 because the adjacent gap (#7) can split the groundwater infiltration from the gap at #1. The seepage rate at #1 for the case of #1 + #3 + #6 exceeded that for the case of #1 + #2 + #7 because #3 and #6 were farther from #1 than #2 and #7. Therefore, #2 and #7 split more water infiltration from #1 than #3 and #6.

5 Hydromechanical coupled behavior at joints

The hydraulic aperture of cohesive elements was fixed in the previous analysis, which is suitable for local leakage areas, such as drainage holes, whose deformation is negligible during operation. However, other leakage areas, such as the segment joints, may be subjected to external loads and deform primarily via relative rotations, which typically occurs in ovalization [43]. This joint behavior may significantly affect the gap flow, as shown in Fig. 2. Next, hydromechanical coupling of the CZM corresponding to the cases shown in Figs. 4(c)–4(e) were considered. In this case, the inserted cohesive elements were used to illustrate the hydraulic deterioration of the

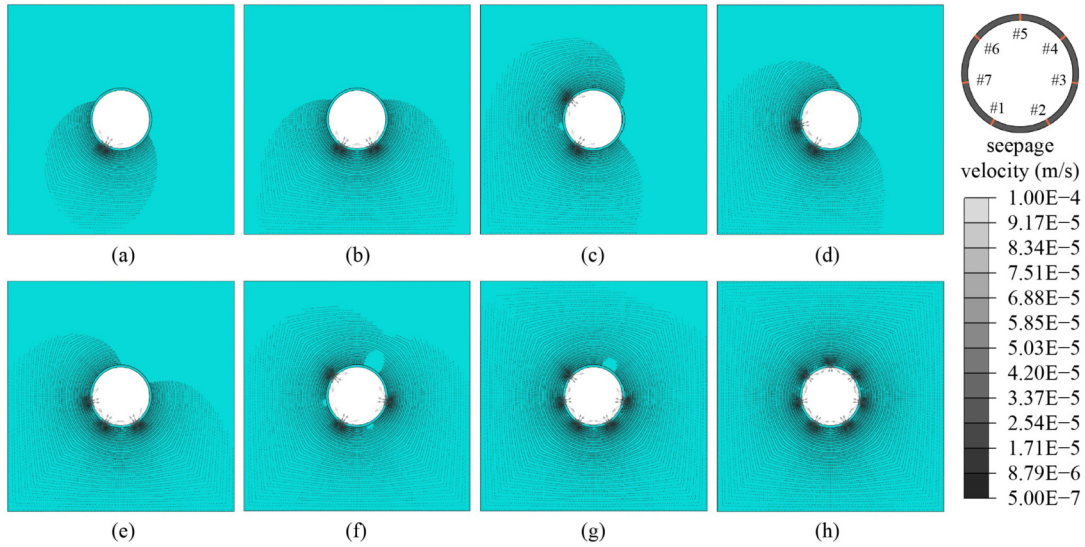


Fig. 15 Seepage velocity for different combinations of leakage gaps: (a) #1; (b) #1 + #2; (c) #1 + #6; (d) #1 + #7; (e) #1 + #2 + #7; (f) #1 + #3 + #6; (g) #1 + #2 + #3 + #6 + #7; (h) #1–#7.

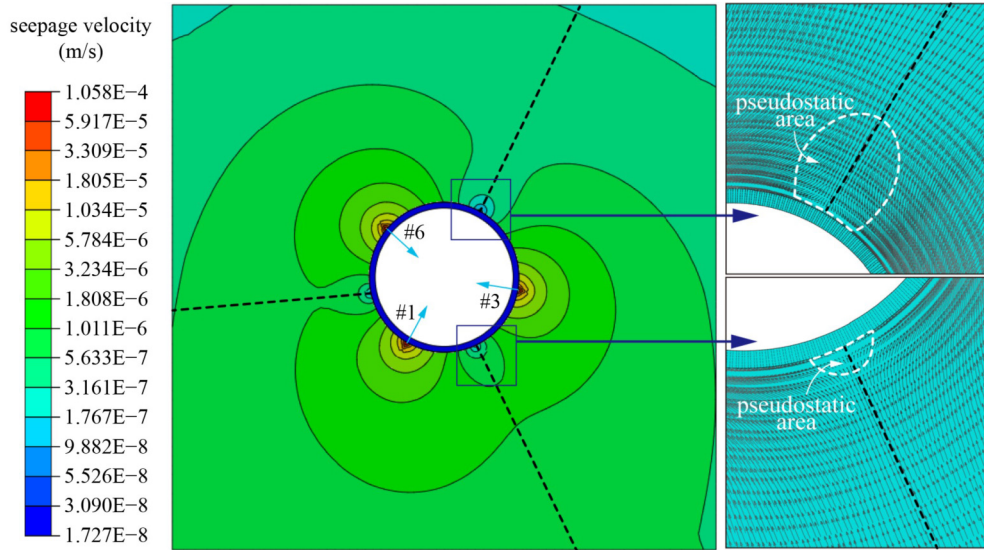


Fig. 16 Influence region of three leakage gaps: #1 + #3 + #6.

joints, whereas the other modeling parameters were the same as those described in Subsection 3.2.

5.1 Single joint deterioration

According to previous studies [32,44], the mechanical behaviors of the joint opening and relative rotation are primarily controlled by the normal stiffness of the interaction elements (cohesive elements in this study) when segments are simulated by solid elements, which subsequently affects the aperture of the hydraulically deteriorated joint. The normal stiffness of the cohesive elements is correlated with the normal modulus E_{nn} . Therefore, a parametric study was conducted by considering five different E_{nn} cohesive elements: 1×10^{10} , 2×10^{10} , 5×10^{10} , 10×10^{10} , and 200×10^{10} N/m². Meanwhile, k_s was set to 1.15×10^{-6} m/s. The initial

aperture g_0 of the joints was set to 0.05 mm to ensure the stable gap flow of the cohesive elements.

Figure 18 shows the effect of E_{nn} on the seepage field with a single joint leakage. Owing to the extrusion of adjacent segments near the joint, the initial gap was compressed owing to joint deformation. The final aperture decreased with E_{nn} . The pore-pressure field was perturbed significantly, particularly near the leakage area. The range of the significantly perturbed pore-pressure field increased with E_{nn} . This is because the seepage rate of a single joint is primarily controlled by the smallest aperture when the two sides of the leakage gap are no longer parallel. To support this statement, the seepage rate and minimum aperture size were plotted against E_{nn} (see Fig. 19). A positive correlation was observed between E_{nn} and the seepage rate. In addition, the seepage rate varied between different joints, even when E_{nn} was

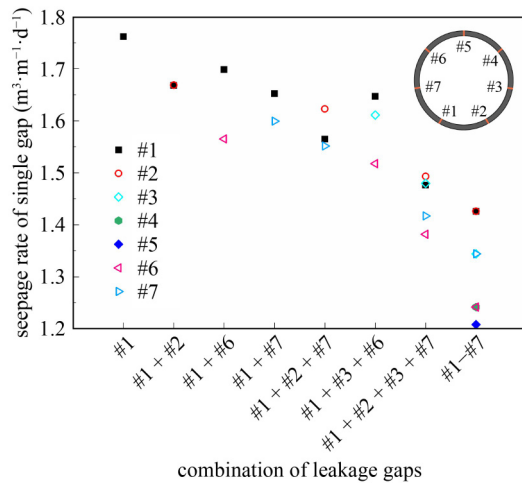


Fig. 17 Seepage rate at single leakage gap under different combinations of gap locations.

set to be identical and the opening was fixed. The seepage rates at gaps #1 and #5 were the highest and lowest, respectively, whereas that at gap #7 was intermediate. This is consistent with the increase in water pressure along the buried depth.

5.2 Effect of buried depth

Owing to the deeply buried case in the aforementioned example, the lining sustained a uniformly distributed high water pressure, which caused the joints to compress. A shallowly buried tunnel is another typical case that is easily affected by surrounding disturbances represented by surface loading [45]. To discuss the effects of different buried depths on the localized hydraulic behavior of the tunnel, a shallowly buried model was established. The buried depth was set to 12 m, and trapezoidal surface loading was applied with an amplitude of 140 kPa to simulate the surrounding disturbance. This additional load was incorporated to exaggerate the joint

deformation. An all-joint deterioration scenario was considered, the g_0 was set to 0.05 mm, and E_{nn} was set to $1 \times 10^{10} \text{ N/m}^2$.

The comparison results presented in Fig. 20 indicate that the deformation patterns were different for the two cases, i.e., the shallowly and deeply buried tunnels. The shallowly buried tunnel exhibited ovalization under surface loading, as shown in Fig. 20(a). The maximum principal stress occurred at the inner side of the tunnel crown and invert, where the joints opened toward the tunnel intrados. By contrast, the joints at the haunch behaved in the opposite manner. However, the deeply buried tunnel contracted uniformly because of the high water pressure, and all the joints were compressed and behaved similarly.

The deeply buried tunnel exerted a more prominent effect on the stratum seepage field and greater groundwater infiltration despite its smaller joint apertures owing to the higher water pressure and the associated higher hydraulic gradient ∇p along the leakage joints. To better illustrate the effect of hydromechanical coupling on water infiltration, the seepage velocities at the external periphery of the lining for the two cases are presented in Fig. 21. The simulation results of the two comparative groups obtained using the local hydraulic conductivity method were overlaid, where k_j was set to $8.8 \times 10^{-7} \text{ m/s}$ via trial calculation based on the results using the CZM without hydromechanical coupling (total leakage amount was 1.7 and $9.2 \text{ m}^3 \cdot \text{m}^{-1} \cdot \text{d}^{-1}$ in the shallowly and deeply buried tunnels, respectively). Based on Figs. 22(a) and 22(b), the local hydraulic conductivity method could not simulate the change in the localized hydromechanical behavior, even when modulus softening (reduced to 10% of the initial lining modulus) was considered at the leakage area. However, when using the CZM, the effect of hydromechanical coupling on seepage behavior was depicted clearly. In the shallowly buried case, when hydromechanical coupling was implemented, the change

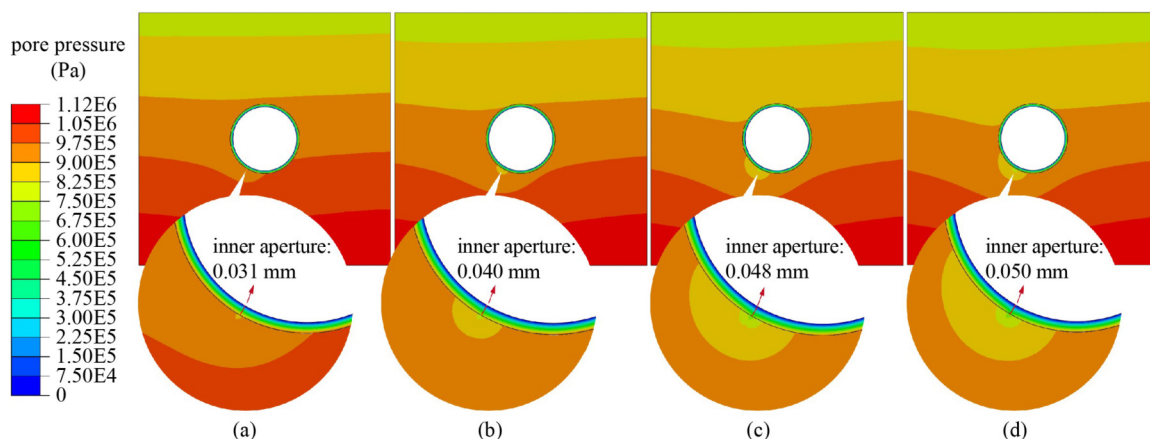


Fig. 18 Seepage fields with different E_{nn} of hydraulically deteriorated joint: (a) $E_{nn} = 1 \times 10^{10} \text{ N} \cdot \text{m}^{-2}$; (b) $E_{nn} = 2 \times 10^{10} \text{ N} \cdot \text{m}^{-2}$; (c) $E_{nn} = 10 \times 10^{10} \text{ N} \cdot \text{m}^{-2}$; (d) $E_{nn} = 200 \times 10^{10} \text{ N} \cdot \text{m}^{-2}$.

in seepage rate varied between joints that experienced different deformations. The seepage velocity decreased from 1.58×10^{-5} to 1.31×10^{-5} m/s at the haunch joint when hydromechanical coupling was considered in the

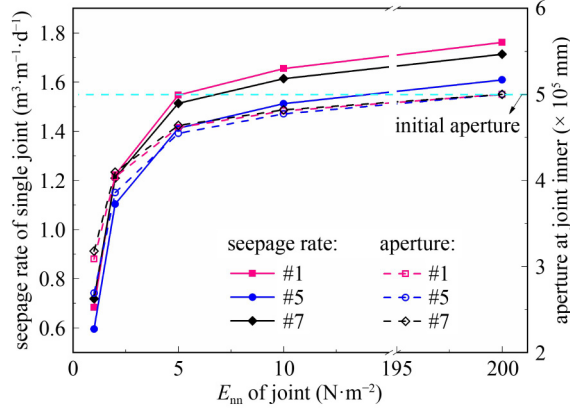


Fig. 19 Seepage rate vs. E_{nn} of hydraulically deteriorated joint.

CZM, whereas it increased slightly at the crown. The effects of hydromechanical coupling on each joint in the deeply buried case were the same owing to their similar compressed states (Fig. 21(b)). The maximum seepage velocity decreased from 8.51×10^{-5} to 4.85×10^{-5} m/s when hydromechanical coupling was considered in the CZM, and the variation was more significant than that for the shallowly buried case.

5.3 Effect of leakage direction

Segmented tunnels have been gradually adopted in pressure tunnels (e.g., water storage and water conveyance tunnels), where localized leakage can occur from the inner to the outer region of the tunnel because of the high internal water pressure (IWP). Hence, we conducted a parametric study that focused on the effect of IWP based on the model presented in Subsection 5.1, where g_0 and E_{nn} were set to 0.05 mm and 1×10^{10} N/m², respectively.

Figure 22 shows the effect of the IWP on the seepage

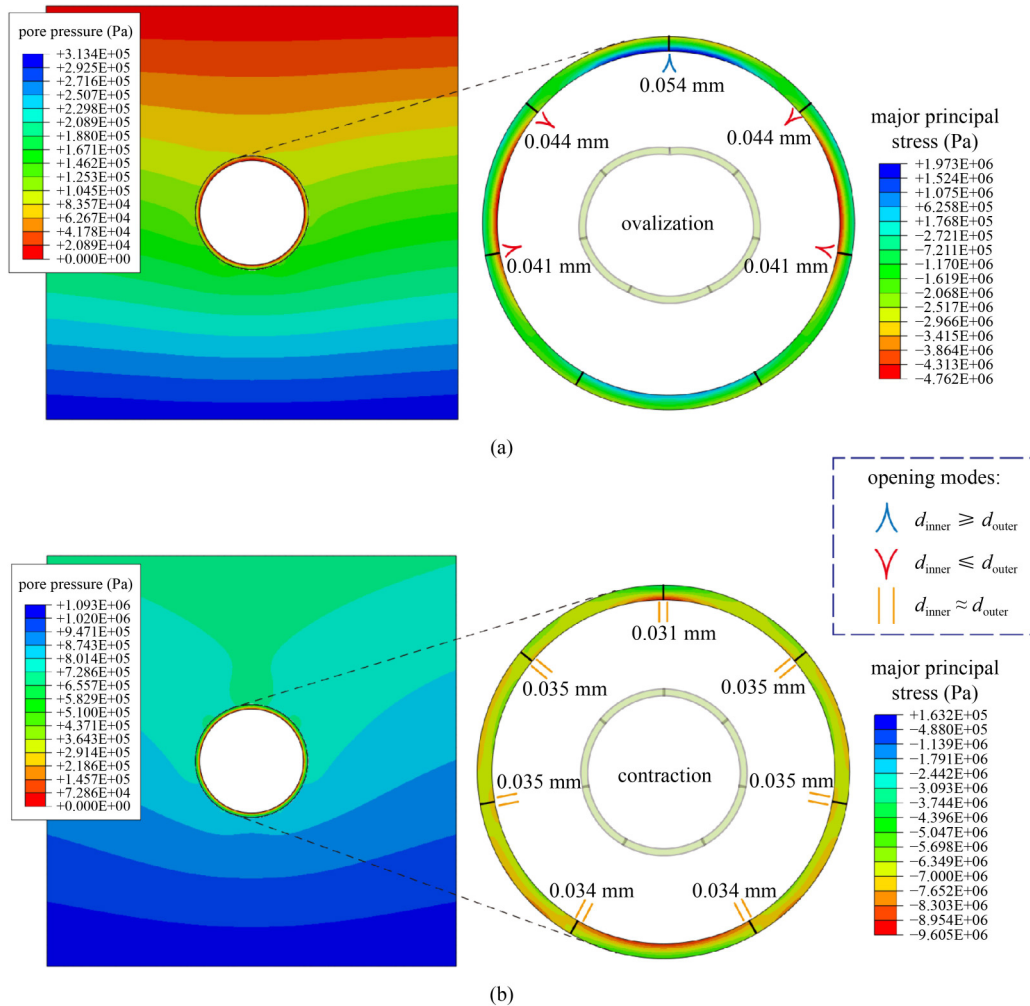


Fig. 20 Seepage field, lining response, and hydraulic opening of shallowly and deeply buried cases: (a) shallowly buried case with surface overloading; (b) deeply buried case with high water pressure.

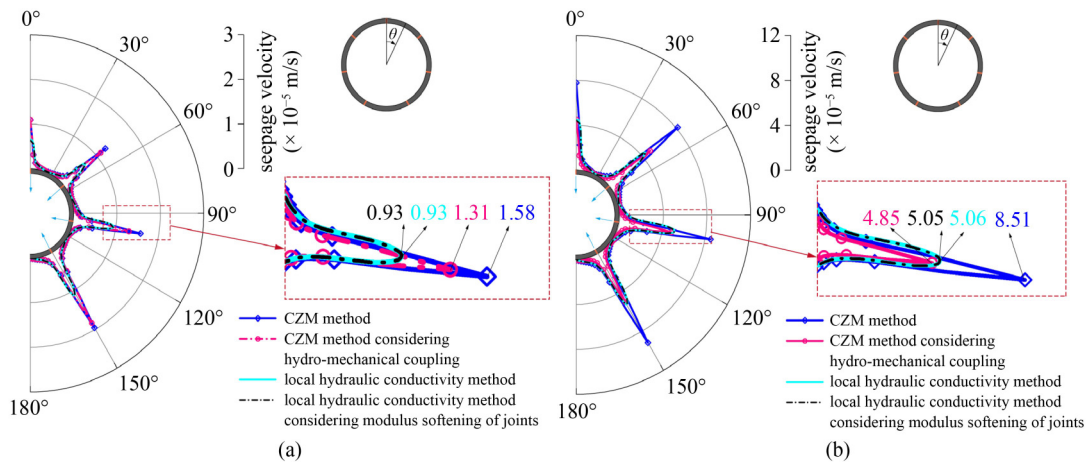


Fig. 21 Effect of hydromechanical coupling on seepage velocity distribution: (a) shallowly buried case; (b) deeply buried case.

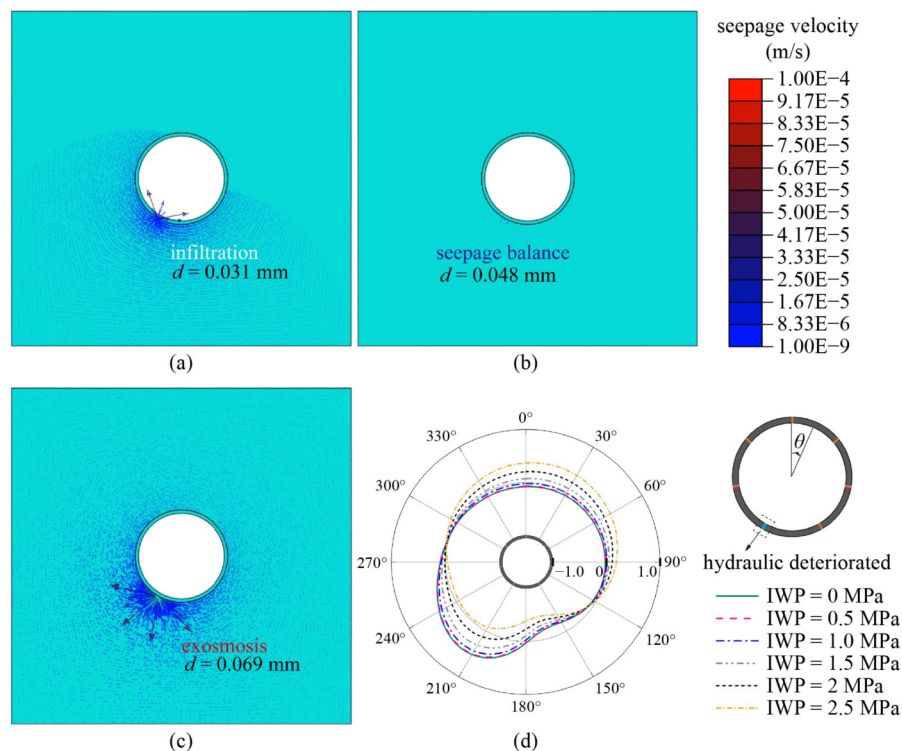


Fig. 22 Effect of IWP on seepage direction and induced response of localized leakage (d represents the final aperture size): (a) seepage fields when IWP = 0 MPa; (b) seepage fields when IWP = 1 MPa; (c) seepage fields when IWP = 2 MPa; (d) effect of IWP on additional radial displacements.

direction and rate, where an IWP range of 0 to 2.5 MPa was considered. As the IWP increased, the variations in the seepage field can be classified into three stages: infiltration, seepage balance, and exosmosis. Seepage balance occurred because the IWP approached the initial ground water pressure. The seepage direction affected the deformation mode of the lining, as shown in Fig. 22(d), and the outward deformation vanished gradually as the IWP increased, resulting in an overall upward trend. The seepage velocity for IWP = 2.0 MPa was significantly higher than that for IWP = 0 MPa, which was primarily due to the variation in the aperture size d . The IWP

reduced the axial forces of the lining, thereby enlarging the d of the hydraulically deteriorated joints.

6 Conclusions

An optimal numerical solution for capturing the hydraulic behavior of localized leakage in segmentally lined tunnels using the CZM was presented herein. The effectiveness of the proposed method was verified by comparing the simulation results with experimental data. A series of parametric analyses was performed while considering the

effects of aperture size, stratum permeability, and leakage point distribution. The importance of coupling the hydraulic and mechanical behaviors of the leakage area when modeling the hydraulic deterioration of segment joints was highlighted. The salient findings of this study are summarized below.

1) Compared with the results obtained using the classical local hydraulic conductivity method, the results yielded by the proposed method were more similar to the experimental data. Moreover, instead of partitioning the localized leakage area with detailed meshes, the CZM discretizes the lining structure by implementing several zero-thickness cohesive elements, whereas the local hydraulic conductivity is derived from their aperture size, thereby improving the calculation efficiency and accuracy.

2) For the single-point leakage case, the effects of stratum permeability and aperture size on seepage behavior were coupled. The effect of stratum permeability was limited when the aperture was small. By contrast, the effect of aperture size was more significant when the stratum permeability was high.

3) Local leakage significantly affected the mechanical response of the lining and surrounding stratum. The seepage perturbed the original stress field near the lining, which was manifested by the formation of a local stress arch near the leakage gap. This resulted in a decrease in ground loading at the leakage gap and an increase in ground loading in the region diametrically opposite to the leakage gap. Additionally, a change in the internal forces was induced within the lining segment, which resulted in an increasing ovalization deformation trend.

4) When multiple leakage gaps existed, the seepage fields were governed by the number and location of the leakage gaps. The average seepage rate of each gap decreased as the number of leakage gaps increased, whereas the bottom gaps indicated a higher average seepage rate than the top gaps owing to the higher water pressure head.

5) The hydromechanical coupled analysis at the segment joints revealed that the seepage rate increased nonlinearly as the joint stiffness increased. This is because a larger joint stiffness corresponds to a smaller joint closure deformation, particularly in the shallowly buried case where the eccentricity is large.

6) The effect of hydromechanical coupling was more evident in shallow tunnels than in deep tunnels. The localized leakage direction may reverse owing to the IWP in pressure tunnels, which consequently affects the lining response. Meanwhile, the IWP reduced the lining axial force, thereby enlarging the joint aperture.

The proposed model provides a promising tool for analyzing the progressive failure of a segmental lining structure subjected to continuous localized leakage under fully hydromechanically coupled conditions. However,

the current model was idealized based on a 2D scenario and cannot reflect the variations in the mechanical response of the lining induced by localized leakage in the longitudinal direction. In the future, more detailed leakage behaviors represented by waterproofing capacity should be investigated.

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