Seismic responses and damage mechanisms of the structure in the portal section of a hydraulic tunnel in rock

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\textbf{ABSTRACT}

Tunnel portals, being shallow underground structures, are well known to be more vulnerable to damage during earthquakes because they are generally located in areas of poor geological conditions that likely lead to amplified ground motions; this is especially true for hydraulic tunnel portals, which are additionally subjected to considerable hydrodynamic pressure. In this work, a full three-dimensional (3D) seismic analysis of the portal structure of a hydraulic tunnel is presented. Additionally, the hydrodynamic pressure and the rock-structure interaction (RSI) are both considered in this analysis to approximate actual field conditions, and their impacts on the seismic response and the damage characteristics of the tunnel portal are studied in detail. The numerical results indicate that the hydrodynamic pressure and the RSI contribute to a greater seismic response of the tunnel structure, with the peak displacements likely increasing by 48% when the hydrodynamic pressure is considered. The tunnel sections as far as 40 m from the entrance may suffer severe damage, and the constraint effects of the surrounding rock can efficiently reduce the damage of the tunnel structure. Two typical damage patterns are deduced from the simulation results and can be reasonably interpreted by the corresponding damage mechanisms.

1. Introduction

The spatial distribution of water resources in China is extremely uneven, resulting in a serious regional water shortage problem. To address this problem, long-distance cross-regional water diversion has become a significant approach. A long-distance water diversion project, in general, must pass through high-seismic-intensity areas; thus, conveyance structures, such as hydraulic tunnels, in those areas are subject to earthquake-induced risks and face severe seismic instability [1]. In particular, water diversion projects have been located in the strong earthquake zone of Southwest China, such as the Water Diversion Project in Yunnan Province, which mainly consists of water-conveyance tunnels with large sections. The safe operation of these tunnels, especially for their portal structures, is severely challenged by strong earthquakes. As the seismically weak part of the tunnel structure, the stability of the tunnel portal is closely related to the safe operation of the entire tunnel [2]. For this reason, it is of great significance to analyze the seismic responses of and probable damage to the hydraulic tunnel portal.

The tunnel portal is a complex geological structure that is strongly affected by the topography of its slope. As a shallow underground structure, the tunnel portal is generally considered to have its aseismic capacity fall between that of a ground structure and that of a deep-buried structure [3,4]. Clearly, compared with the strong restraint of the surrounding rock on a deep-buried structure, the restraint effect is much weaker on the portal structure because the surrounding rock here is generally highly weathered or even completely broken; in addition, the ground motion is amplified by the free surface of the slope, resulting in a greater seismic inertia [5]. Both of these parameters are the major factors that contribute to portal instability. According to earthquake damage investigations, a number of tunnel portals have suffered damage, ranging from slight cracking to severe collapse, during several recent strong earthquakes [1,6–9]. Dowding and Rozen [10] collected 71 case histories of tunnels, among which, 42 tunnels experienced varying levels of damage during 13 earthquakes with magnitudes ranging from 5.8 to 8.3. Wang et al. [1] conducted a systematic investigation of tunnel damage resulting from the Taiwan Chi-Chi Earthquake ($M_w = 7.6$) and demonstrated that portal failure is one of the most common damage patterns for tunnels. Yashiro et al. [11] proposed a simple classification method for seismic damage patterns of...
2. A brief description of the Fengtun tunnel

2.1. Location and engineering geology of the tunnel portal

The Water Diversion Project in Yunnan Province of China, a large-scale water conservancy project, is mainly used to alleviate the regional water shortage in Yunnan province, and its water diversion structure mostly consists of hydraulic tunnels. The Fengtun diversion tunnel studied in this paper, as the experimental program of the project, is located in Chuxiong, Yunnan Province, with a total length of 9829.4 m (see Fig. 1). The terrain slope angle of the tunnel entrance is approximately 30° – 40° with a thin and severely developed overlying rock mass. The tunnel portal passes through two formations: the Quaternary-aged conglomerate near the slope surface, which is composed of clay and crushed rock with a thickness of 2–3 m, and sublayers that are generally strong or weakly weathered Cretaceous sandy mudstones (see Fig. 2).

Fig. 2 shows the horizontal vertical section of the tunnel portal. The geological units of the tunnel portal mainly consist of calcareous sandy mudstone, muddy sandstone and other soft rock. The geomechanical properties of the rock units are obtained by in situ tests and given in Table 1.

2.2. Structure type and support of the tunnel portal

The cross section of the Fengtun tunnel portal structure has a total width of 8 m and a maximum height of 10 m, and the designed water depth of the tunnel is 7.51 m (see Fig. 3). The tunnel was excavated using the New Austrian Tunneling Method. Because the surrounding rock of the portal section mainly consists of highly weathered soft rock that contributes to a weak bearing capacity, it is necessary to take support measures. The support design mainly consists of slope support and tunnel support. For the slope of the portal, an artificial excavation with a 60 m excavation height and a 1:0.75 excavation slope ratio is conducted. For the tunnel, a primary and a secondary support are adopted. The primary support consists of 0.2 m of primary lining, steel arches and rock bolts. The rock bolts, with lengths of 6 m and diameters of 25 mm, are distributed on a grid of 2.0 m × 2.0 m for weakly weathered rock masses and 1.5 m × 1.5 m for strongly weathered rock masses. The tensile strength of rock bolts is 360 MPa. The secondary support is composed of reinforced concrete with a thickness of 0.6 m. The concrete used for the tunnel support, including the primary lining and secondary lining, is C25-grade [15]. Note that the groundwater is not considered in this work because the portal structure is above the groundwater level. Details of the support parameters are presented in Table 1.

2.3. Seismicity of the study site

The Fengtun diversion tunnel is located in the Xianshuihe fault zone, where the seismic activity is strong and of high frequency. As one of the most intense regions of modern tectonic activity and major strong earthquake activity areas in China, the earthquake activities in this area are mainly controlled by intensive fault zones. According to devastating earthquake records from 1623 AD, there have been 21 earthquakes with magnitudes greater than 4.7 in the near field of the tunnel, of which a magnitude between 6.0 and 6.9 has occurred 4 times. Moreover, according to the seismic ground motion parameter zonation map of China [16] prepared by the China Earthquake Administration, the tunnel portal studied in this work is located in an area with an expected basic intensity of seven degrees. To examine the seismic resistance of the tunnel portal, a dynamic response simulation is conducted in the next section.

3. Numerical model

3.1. Finite element model

Three methods are mainly adopted for the seismic response analysis of underground structures: damage observation, model tests and numerical analysis. The real seismic response of the structures can be obtained by damage observation after earthquakes; this response is significant for the development of the seismic theory of underground structures. However, the occurrence of earthquakes is usually accidental, and the current seismic data are still relatively scarce and cannot meet the growing demand for engineering seismic design. Detailed structural seismic response data can be obtained by model tests, but the method is limited by experimental conditions as well as monitoring means. Numerical analysis can simulate complex boundary conditions and is the most widely used method for seismic response analysis. Among the numerical methods, the finite element dynamic time-history method is relatively effective. Based on an in-house dynamic FEM numerical simulation platform (SUCEED) [17], a full 3D dynamic time-history analysis of the Fengtun diversion tunnel portal is carried out in an attempt to investigate its dynamic response. A limited computing model is intercepted to establish a 3D finite element model, which is composed of 64244 eight-node hexahedral elements (see Fig. 4). The maximum mesh size is set as 5 m to meet the requirement of dynamic calculation accuracy [18]. To absorb the energy of scattered
3. Material model

3.2. Plastic damage model for concrete material

In this paper, the plastic damage model established by Lee et al. [20] is adopted to simulate the mechanical properties of concrete under seismic cyclic loadings for two reasons: (1) it can account for the accumulated damage effect of concrete material during cyclic loadings, and the stiffness degradation when cracking occurs; and (2) its simple form contributes to the large-scale finite element calculation, and its applicability has been extensively verified by static and dynamic analyses. The model can be described briefly as

\[
\sigma = (1 - d)\sigma_0 - (1 - d)D_0; \quad (\varepsilon - \varepsilon^p)
\]

\[
\dot{\varepsilon}^p = \frac{\dot{\lambda}G(\sigma)}{\Delta \sigma}
\]

where \(\sigma\) represents the effective stress tensor; \(D_0\) is the initial elastic matrix; \(\varepsilon^p\) denotes plastic strain tensor; \(\lambda\) is the plastic multiplier; \(G\) is the Drucker–Prager hyperbolic function and is adopted as the plastic flow potential; \(d\) is the scalar damage variable and can be evaluated by interpolating between tension damage variable \(d_t\) and compressive damage variable \(d_c\); \(s_t\) and \(s_c\) are introduced to describe stiffness recovery effects during load reversal and are defined as

\[
\begin{align*}
s_t &= 1 - w_t r(\sigma) \\
s_c &= 1 - w_c r(\sigma)
\end{align*}
\]

\[
r(\sigma) = \frac{\sum_{i=1}^{3} \langle \sigma_i \rangle}{\sum_{i=1}^{3} |\sigma_i|}
\]

where \(w_t\) and \(w_c\) are weight factors that control the stiffness recovery degree; \(r(\sigma)\) is the stress weight factor; \(\langle \cdot \rangle\) denotes the McCauley bracket.

\[
d = 1 - (1 - s_t d_t)(1 - s_c d_c) \quad 0 \leq s_t, s_c \leq 1
\]

Fig. 1. Location of the Fengtun tunnel.

Fig. 2. Longitudinal vertical section of the Fengtun tunnel portal.
Generally, the water in the hydraulic tunnel with a large cross section is of considerable mass, and its impacts on the seismic response of tunnel structure cannot be ignored [21]. The dynamic interaction between the internal water and the tunnel structure is an extremely complicated nonlinear process; thus, it is usually necessary to simplify this process first. A simplified added mass method proposed by Westergaard [22] is adopted in this paper to simulate the hydrodynamic pressure. The internal water is equivalent to an additional mass acting on the tunnel structure in the method, and the hydrodynamic pressures at certain positions of the tunnel structure are assumed to be inertial loads caused by the movement of a certain mass of fluid added at each position. Moreover, the internal water and tunnel structure are modeled independently, and the numerical simulation of inertial actions of internal water on tunnel structure is achieved by superimposing the additional mass matrix to the motion equations. Since this method is decoupled, it is easy to apply to large-scale finite element calculations. However, it is clear that the effects of water sloshing are not taken into account in the added mass method. In fact, the shaking of a large volume of water will clearly affect the mechanical behaviors of the tunnel structure. Nevertheless, because the focus of this paper is to study the seismic responses and damage mechanisms of the portal structure, it is acceptable to carry out numerical simulations using this simplified method. Added mass $M_w$ can be calculated by the following equation [23]:

$$M_w(z) = \frac{7\rho_w \sqrt{\gamma z}}{8}$$

(6)

### Table 1

<table>
<thead>
<tr>
<th>Material</th>
<th>Highly weathered mudstone</th>
<th>Moderately weathered mudstone</th>
<th>Slightly weathered mudstone</th>
<th>Primary lining</th>
<th>Secondary lining</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (g/cm³)</td>
<td>2.4</td>
<td>2.5</td>
<td>2.7</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Deformation modulus (GPa)</td>
<td>0.5</td>
<td>1.0</td>
<td>3.0</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>Elastic modulus (GPa)</td>
<td>~</td>
<td>~</td>
<td>~</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.27</td>
<td>0.25</td>
<td>0.23</td>
<td>0.167</td>
<td>0.167</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>16.7</td>
<td>21.8</td>
<td>35</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Dilation angle (°)</td>
<td>~</td>
<td>~</td>
<td>~</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>0.1</td>
<td>0.2</td>
<td>0.7</td>
<td>0.666</td>
<td>0.666</td>
</tr>
<tr>
<td>Eccentricity</td>
<td>~</td>
<td>~</td>
<td>~</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>$K_c$</td>
<td>~</td>
<td>~</td>
<td>~</td>
<td>0.666</td>
<td>0.666</td>
</tr>
<tr>
<td>Stress ration</td>
<td>~</td>
<td>~</td>
<td>~</td>
<td>1.16</td>
<td>1.16</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>0.3</td>
<td>0.6</td>
<td>1.5</td>
<td>1.1</td>
<td>1.2</td>
</tr>
<tr>
<td>Compressive yield stress (MPa)</td>
<td>~</td>
<td>~</td>
<td>~</td>
<td>10.5</td>
<td>11.7</td>
</tr>
<tr>
<td>Uniaxial Compressive strength (MPa)</td>
<td>5</td>
<td>10</td>
<td>25</td>
<td>16.7</td>
<td>20</td>
</tr>
</tbody>
</table>
where $\rho_f$ is the fluid density; $h$ is the water depth of the tunnel; $z$ is the distance from the calculation position to the water surface; $\eta$ is the reduction factor and can be valued as 0.66.

### 3.2.3. Dynamic interaction analysis model for tunnel and surrounding rock

The contact effect between the surrounding rock and the tunnel structure that increases with the relative rigidity of the two has a great influence on the seismic response of the tunnel structure [23]. The deformation modulus of the concrete lining is nearly 10 times that of the surrounding rock in this work (see Table 1); therefore, the effects of dynamic interaction must be considered. However, the rock-tunnel interactions are complicated by the nonlinear mechanical characteristics of the contact system caused by their complex contact states. In fact, there are three contact states, namely, bonding contact, sliding contact and separation, between the surrounding rock and the tunnel [24]; they all should be considered in an appropriate dynamic contact model.

Fig. 5 shows the contact system and the dynamic contact forces on the contact face. According to Liu et al. [25], the contact system $S_i$ can be divided into two sides of surface $S_i$ for the surrounding rock and $S_j$ for the tunnel structure, on which the correspondent nodes, i.e., $i$ and $i'$, are node pairs. The two nodes of each node pair occupy identical coordinates and their node forces satisfy the interaction relationships of the normal and tangential components of the nodal displacements along the contact face. Combining Eqs. (8) and (9), the contact force can be given by

$$
H_i(U_{i'}^{n+1} - U_i^{n+1}) = \begin{bmatrix}
U_{i1}^{n+1} - U_{j1}^{n+1} \\
U_{i2}^{n+1} - U_{j2}^{n+1} \\
U_{i3}^{n+1} - U_{j3}^{n+1}
\end{bmatrix} = \begin{bmatrix}
\frac{2M_i}{M_i + M_j} \Delta t M \cdot \Delta U_i^{n+1} \\
\frac{\Delta t}{M_i + M_j} \Delta t M \cdot \Delta U_i^{n+1} - \frac{\Delta t}{M_i + M_j} \Delta t M \cdot \Delta U_i^{n-1} + \frac{\Delta t}{M_i + M_j} \Delta t M \cdot \Delta U_i^{n-1}
\end{bmatrix}
$$

where $U_{i1}^{n+1}$ and $U_{i2}^{n+1}$ are displacement vectors of nodes $i$ and $i'$ in the global coordinate system; $H_i$ is the transformation matrix for the global and local coordinate system; and the subscripts 1, 2 and 3 denote the tangential ($u_t$) and normal ($\sigma_n$) components of the nodal displacements along the contact face. Combining Eqs. (8) and (9), the contact force can be given by

$$
\begin{bmatrix}
T_i^n \\
T_j^n
\end{bmatrix} = \frac{2M_i M_j}{(M_i + M_j) \Delta t^2} \begin{bmatrix}
(U_{i1}^{n+1} - U_{j1}^{n+1}) \\
(U_{i2}^{n+1} - U_{j2}^{n+1}) \\
(U_{i3}^{n+1} - U_{j3}^{n+1})
\end{bmatrix}
$$

where $T_i^n$ and $T_j^n$ are subvectors of the tangential force $T_i^n$; therefore, $T_i^n = T_j^n + T_3^n$. Clearly, the establishment of Eq. (10) is based on the assumption that the surrounding rock and the tunnel structure are strongly bonded together. However, sliding and separation may occur between the two; thus, it is important to check the contact forces $N$ and $T$ as follows:

$$
\begin{bmatrix}
N_i^n = T_i^n = 0 \\
N_j^n = T_j^n = 0 \\
N_i^n = 0 \land N_j^n \leq 0 \land N_j^n > c_i A_i \\
N_j^n = 0 \land N_j^n > c_i A_i \\
T_i^n = \mu N_i^n ||T_i^n||/||T_i^n|| \land N_j^n \leq 0 \land ||T_i^n|| > A_i ||N_i^n|| + c_i A_i
\end{bmatrix}
$$

where $\mu_i$ and $\mu_j$ are the static and kinetic friction coefficients, respectively. $c_i$ and $c_j$ represent the initial cohesion and tensile strength of the contact face, respectively. $A_i$ is the control area of node $i$. In denoting “and”. According to Eqs. (10) and (11), the contact force can be calculated and checked by substituting the contact force into Eq. (8), the motion state of the contact system at time $n + 1$ can be updated accordingly.

### 3.2.4. Modeling procedure

To approximate the actual construction process and make the numerical simulation more credible, the simulation process is divided into 3 steps. In the first step, the gravity field is adopted as the initial stress field. The vertical stress $\sigma_z$ induced by gravity varies linearly with depth. The horizontal stress $\sigma_x$ is calculated by $\sigma_x = k \sigma_z$, where $k$ is the lateral pressure coefficient. According to the on-site investigation of the designer of the Fengtun tunnel, the lateral pressure coefficient is recommended to be 1.4.

In the second step, static excavation calculation is conducted. In this process, the first and second supports are considered. The first supports mainly include rockbolts, the steel arch and the primary lining. The reinforcing effect of the rockbolts is simulated by an implicit cylindrical anchor bar element proposed by Xiao et al. [27]. The support effects of the steel arch and primary lining are considered by an equivalent method, in which the steel arch is not modeled separately, with the method only considering their strengthening effects on the elastic modulus of the primary lining concrete; thus, the equivalent elastic modulus $E_{eq}$ of primary lining can be given by

$$
E_{eq} = E_e + \frac{S_e E_e}{S_c}
$$

where $E_e$ and $E_l$ are the elastic modulus of the primary lining and steel arch, respectively. $S_e$ is the cross-sectional area of one grille steel arch. $S_c$ is the control area of one grille steel arch. The second support is the secondary lining. A relatively simple method by Chen et al. [28] is employed to simulate the supporting effects of secondary lining, and perfect bonding is assumed between the rock and the lining at this
stage. In addition, the joint bearing effect of the concrete lining and its reinforcement is considered by the method of Bian et al. [29].

In the third step, the stress state computed in the second step is used as the initial condition for subsequent seismic analysis. In the process of seismic analysis, the behaviors of the rock units are assumed to be elastic-plastic damage and obey the Mohr-Coulomb failure criterion with tension cut-off [17], and the plastic damage model [20] introduced in Section 3.2.1 is used for the concrete medium. Moreover, the viscous spring boundary by Liu et al. [30] is also imposed on the lateral and bottom boundaries of the model, and the Rayleigh damping is adopted with a critical damping ratio of 5%. According to site conditions and seismic design standards of the hydraulic tunnel, the input artificial seismic wave is shown in Fig. 6. Three motion directions are considered: first is the horizontal X direction with a maximum amplitude of 2.94 m/s² (see Fig. 6a), which is transverse to the longitudinal axis of the tunnel (see Fig. 4); second is the horizontal Y direction with a maximum amplitude of 2.45 m/s² (see Fig. 6b), which is parallel to the longitudinal axis of the tunnel; third is the vertical Z direction (see Fig. 6c) and its maximum amplitude is 1.96 m/s². All motions are applied on the bottom boundary of the model and incident upward.

4. Results and discussions

This work aims at the seismic analysis of the Fengtun tunnel portal, and the detailed analysis process in this section is given as follows. First, two cases with internal water in the tunnel being considered or not are contrasted to investigate the impacts of hydrodynamic pressure; then, two comparative studies, one with RSI and the other not, are performed to estimate the RSI effects on tunnel structure. The case without RSI means that the tunnel and surrounding rock are assumed to be tied together during modeling at the rock-tunnel interface, and the dynamic interaction of the rock and tunnel is not considered. Eventually, the damage mechanism of the tunnel portal is analyzed based on the above research. More specifically, to illustrate the calculation results, three typical cross sections (S1–S3) are chosen for analysis; their arrangements are shown in Fig. 7a. In each section, five monitoring points are selected, and their locations are given in Fig. 7b. These points are representative of crown (A), spandrel (B), sidewall (C), base corner (D), and floor (E).

4.1. Impacts of hydrodynamic pressure on the seismic responses

4.1.1. Stress response of the tunnel portal structure

The hydrodynamic pressure can be calculated and addressed at the secondary lining by the added mass method introduced in Section 3.2.2. Cross section S1 (see Fig. 7) is chosen as the most critical section to analyze the stress response of the secondary lining under the following two cases: the internal water of the tunnel is not considered in case 1, and the internal water is considered in case 2. The maximum principle stress is used to estimate the damage of the tunnel structure. The
magnitude of the maximum principal stress, which can be considered as the indicator of damage, is of primary concern because the concrete material is well known to have weak tensile capacity. As shown in Fig. 8, the maximum principal stresses at the crown and spandrel are greater than those of the other positions in the two cases, and their magnitudes even exceed the tensile strength of concrete, which is denoted by the black, dashed, and horizontal lines in Fig. 8. This result indicates that tension cracks are likely to occur on the crown and spandrel (points A and B). Moreover, note that the magnitudes under the scenario of case 2 are approximately 17.1% greater than those of case 1, and the closer to the entrance, the greater the tension stress suffered at the secondary lining (see Fig. 9). We can conclude from the
above results that when internal water is considered, the stress response is more extensive than that of case 1. Clearly, the different results of the two cases are mainly caused by the hydrodynamic pressures, which act as additional loads on the secondary lining and consequently induce a greater stress response. Hence, it is essential to consider the effects of hydrodynamic pressure in the aseismic design of a hydraulic tunnel.

4.1.2. Displacement response of the tunnel portal structure

Only the X and Z direction displacements are discussed here. Fig. 9 displays the displacement time-history curves of the monitoring points at section S1 under the two cases. Clearly, the waves of the curves are roughly similar to each other, and their peak values appear nearly at the same time, indicating that the displacement fluctuation is mainly controlled by the input earthquake waves. Comparing the two cases, the major difference lies in the displacement amplitudes. When the internal water is considered, the amplitudes of the X displacements are nearly 2.3 cm larger than those without water, and those of the Z displacements are approximately 2.8 cm larger than those without water. The responses of different positions of the tunnel are also evidently varied in terms of displacement amplitudes; thus, structural deformation occurs. As shown in Fig. 10, the maximum relative displacements are 3.47 cm in the X direction and 2.80 cm in the Z direction in case 2; they are larger than that of case 1. Fig. 11 shows the peak displacements of the crown (point A) at the three sections. The two cases show similar results. The values of the peak displacements in the two directions decrease along sections S1 to S3 in both cases. In addition, the difference of the results in the two cases is also obvious, as the peak values in case 2 are much greater than the peak values in case 1. The hydrodynamic pressures contribute to a 30%–48% increase in the peak displacements. Therefore, it can be stated that the hydrodynamic pressures induce a larger displacement response and cannot be neglected.

4.2. Impacts of RSI on the seismic responses

The plastic strain of the tunnel structure is adopted to gain insight into the impacts of RSI on the seismic response in this section. Fig. 12a and (b) plot the equivalent plastic strain contours of the tunnel structures with or without RSI. The results are plotted every 5 s during the calculation; the plots show similar distributions of plastic strain. After static excavation calculation (t = 0 s), the values of plastic strains at the spandrels and floor corners are much greater than the values at other locations, and the maximum occurs at the right floor corner with a somewhat larger plastic yielding zone. Such inconsistency of the plastic strains suggests unignorable structural deformation, in agreement with the results offered in Section 4.1. The extent and degree of plastic yielding increase with time, and most positions are occupied by the yielding zone at the end of the calculation. It is interesting that the spandrels and the floor corners suffer larger plastic strain than other positions, which means these positions are the weak parts of the tunnel lining. Tao et al. [31] conducted a model test to study the seismic response of tunnel portals and observed similar phenomena; thus, the results of this paper are credible. In addition, the comparison of the results of the two cases shows that the RSI results in a greater plastic strain response because the maximum value of the plastic strain and the range of plastic yielding in the case with RSI are clearly greater than those without RSI. Moreover, the internal forces of the tunnel structure in the case of RSI are also larger than those without RSI (see Fig. 13). The main reason for the difference is that while the RSI is not considered, the tunnel structure and the surrounding rock are assumed to be tied together, and there is a strong restraint effect to reduce the seismic damage of the tunnel structure. A similar conclusion can also be found in the work of Zlatanović et al. [14].
4.3. Seismic damage characteristics of the portal tunnel structure

4.3.1. Damage distribution of the tunnel lining and the contact face

The damage coefficient $d$ of the concrete structure, which can be obtained by Eq. (3), is employed to quantitatively display how much seismic damage is induced and where it is severely damaged. In this section, the damage coefficient distribution of the tunnel with internal water and the RSI being considered is depicted in Fig. 14. Fig. 14 shows that the tunnel damage increases with time. After the static excavation calculation, the tunnel lining within 5 m of the entrance suffers severe damage ($d > 0.6$). When the time is 5 s, the severely damaged positions, where damage coefficients are greater than 0.7, are mainly located at the crown, spandrels and floor corners of the lining and extend a distance of 10 m from the entrance. This phenomenon is consistent with the results plotted in Fig. 12. With increasing time, the tunnel damage continues to spread along the longitudinal axis direction. At the end of the simulation, namely, at $t = 20$ s, the tunnel reaches the most severe damage, and areas as far as 40 m from the entrance may suffer severe damage. In addition, it can also be observed that the closer to the entrance, the more severe the tunnel damage is.

Generally, the tunnel lining is strongly bonded to the surrounding rock via grouting effects before contact face sliding or cracking occurs. Thus, the tunnel and the surrounding rock can be regarded as a whole, and the RSI can be ignored if damage occurs on their contact face. However, it is difficult for the surrounding rock and tunnel lining to deform in synchrony under seismic actions, and the damage to the contact face is difficult to avoid. In fact, the spalling of a tunnel from surrounding rock is very common according to in situ damage investigations of tunnels [1,12]. Fig. 15 plots the damage distribution of the contact face at the end of the static and dynamic simulations. At the end of the static excavation calculation ($t = 0$ s), the crown and part of the sidewalls within 5 m of the entrance suffer slip or crack damage. This damage indicates that the excavation process can also lead to damage to the contact face. When the time is 20 s, the damaged areas are mainly at the crown and the sidewalls near the entrance, with a tendency toward further development along the longitudinal axis.
direction. Moreover, when considering Figs. 14 and 15, it is obvious that the damage distributions of the contact face are consistent with that of the tunnel lining, suggesting that a good contact state is significant for the stability of the tunnel lining, and the constraint effects of the surrounding rock to the lining can efficiently reduce the seismic damage of the tunnel structure [24].

4.3.2. Damage mechanism of the portal tunnel lining

Based on the results of the simulation, two types of damage patterns of the tunnel structure, namely, damage due to structural deformation and high squeezing loads, are classified for a reasonable explanation of damage mechanisms of the Fengtun tunnel portal under strong earthquakes. Fig. 16 and Fig. 17 provide comparisons between the predicted results from the model, shown in Figs. 16 (a) and Fig. 17 (a), with their damage mechanisms, shown in Figs. 16 (b) and Fig. 17 (b). Note that, as shown in Fig. 16a, the displacement of the tunnel structure follows a general tendency to gradually decrease from the crown to the floor, and the upper structure of the tunnel suffers the largest displacement. This phenomenon is a typical characteristic of portal structures under earthquakes. The relative deformation of the upper and lower structures is mainly generated by amplified horizontal forces. The horizontal forces acted on tunnel lining contribute to considerable bending moments at the spandrels; as a result, tension cracks are generally likely to occur at these positions under strong earthquakes (see Fig. 16 (b)). This damage mechanism can also be observed in the previous works of Yashtiro et al. [11] and Shen et al. [5]. In addition, Figs. 12 and 14 show that the spandrels are parts of the most severely damaged positions, corresponding with the above damage mechanism.

Fig. 17a shows a contour plot of the Z-direction displacements of the other positions relative to position A. Clearly, the tunnel floor suffers the maximum displacement, and a significant uplift, which is frequently observed at the tunnel portal, occurs at the center floor with a value of nearly 24 mm. Tunnel uplift damage is a complex process and could be influenced by various factors during earthquakes [32]. However, focusing on the tunnel in this work, two factors are considered the main causes of the damage: one is the poor geological condition of the tunnel,
and the other is the high squeezing loads caused by amplified seismic motions. The tunnel portal is likely to be affected by an earthquake because it is constructed in weak rocks with low strength and causes high squeezing loads on the tunnel; thus, the squeezing loads are exerted on the sidewalls and floor and cause high tensile stress and buckling deformation on the floor. As a result, uplift damage is likely to appear here.

5. Summary and conclusions

The portal section of the Fengtun hydraulic tunnel is located in poor geological conditions and suffers the risk of strong earthquakes. What makes this case study special are the large amounts of water in the tunnel and the nonignorable RSI effects, both of which are considered to result in greater seismic responses or even seismic damage to the tunnel structure. Therefore, for a more accurate simulation, the effects of the internal water and the RSI are both considered in a full 3D FEM model, and then the damage characteristics and mechanisms of the portal tunnel structure are analyzed in detail. The following conclusions can be obtained from the research:

(1) The hydrodynamic pressure caused by seismic inertia force has a significant effect on the performance of the tunnel because it acts as an additional load on the tunnel and induces greater stress and displacement responses; the peak displacements are likely to increase by 48%.

(2) In the case with the RSI, the maximum value of the plastic strain and the range of plastic yielding of the tunnel section are clearly greater than those without the RSI, meaning that the RSI has an essential influence on the tunnel response.

(3) The damage characteristics of the tunnel structure are analyzed considering the effects of internal water and the RSI. The results unequivocally show that the damage continues to extend along the cross section and the longitudinal axis direction during the earthquake, and the closer to the entrance, the more severe the tunnel damage. In addition, the constraint effects of the surrounding rock on the lining can efficiently reduce the damage to the tunnel structure.

(4) Two typical damage patterns, namely, damage due to structural deformation and squeezing loads, are classified, and their damage mechanisms are analyzed in detail. The structural deformation of the tunnel is mainly the result of amplified horizontal forces and contributes to the bending cracks on the spandrels. The tunnel uplift is generally caused by high squeezing loads acting on the sidewalls and floor of the tunnel.

Fig. 15. Damage zone of contact face.

(a) X direction displacement of tunnel lining at section S1 at t = 20 s

(b) Sketch of damage mechanism (from Yashiro, et al. [11])

Fig. 16. Tunnel damage due to structure deformation.

(a) Relative Z-direction displacement of tunnel lining at section S1 at t = 20 s

(b) Sketch of damage mechanism

Fig. 17. Tunnel damage due to large squeezing loads.
Conflicts of interest

The authors declare that there is no conflict of interests regarding the publication of this paper.

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Appendix A. Supplementary data

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References
