Seismic stability assessment of the portal of a hydraulic tunnel under obliquely incident P waves

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Abstract. Tunnel portals are one of the most vulnerable sections that suffer the risk of instability during earthquakes because they are generally located in poor geological conditions. What makes this case studied in this work even more special are the oblique incident seismic waves and the large amounts of water in the hydraulic tunnel, both of which are believed to cause more complex mechanical behaviors and larger seismic response of hydraulic tunnels. In this paper, a full three-dimensional finite element model that takes the oblique incident seismic waves and hydrodynamic pressure into account is established to evaluate the stability of the tunnel portal under different incident angles of seismic waves. The numerical results indicate that: (1) the stress and displacement responses induced by oblique incident P waves are much larger than that by horizontal and vertical incident P waves; (2) the closer to the tunnel portal, the larger seismic response of the tunnel; (3) tunnel portal is where damage occurs first and suffers severe damage; (4) both oblique incident seismic waves and hydrodynamic pressure are likely to cause larger damage zone and more severe damage.

Keywords. Hydraulic tunnel; Portal structure; Seismic performance; P waves; Obliquely incident.

1. Introduction

It is traditionally acknowledged that tunnel portals, which although have a limited area of influence, are more vulnerable to damage than other positions of tunnels during earthquakes, and their damage are closely associated with structural stability of tunnels[1-3]. The weak seismic stability of tunnel portals is believed to be due primarily to the fact that their overburden, which mostly consists of heavily weathered or even completely broken rocks, is too thin and of weak strength to offer enough constraint on tunnels; and as a result, slope instability and tunnel collapse are likely induced by external load disturbance[4]. Additionally, seismic motion will be amplified by the slope surface of tunnel portals, which further increases the adverse effects of seismic loads on the stability of tunnel portals [5,6]. Therefore, it is of utmost importance to study the seismic behaviors of tunnel portals and improve their seismic stability.
The failure of tunnel portals have been observed and reported during several famous strong earthquakes, including the 1989 Loma Prieta earthquake [2,7], 1995 Kobe earthquake [8,9], 1999 Chi-Chi earthquake [10,11] and 2008 Wenchuan earthquake [12-14], and since then the seismic behaviors of tunnel portals therefore becomes an hot topic and arouses much attention. Numerous researchers have investigated the seismic response of tunnel portals. Dowding and Rozen [15] investigated the seismic behaviors of 71 tunnels, among which 42 tunnels were observed suffered varying levels of damage during earthquakes. Sharma and Judd [16] collected 192 reports of seismic behavior of tunnels and found 94 tunnels suffered damage. Wang et al. [11] conducted a statistics of tunnel damage in the Chi-Chi earthquake. They concluded portal failure is one of the most common failure patterns. Shen et al. [5] investigated the damage characteristic of 52 tunnels after the Wenchuan earthquake and 13 tunnel portals were reported to suffer severe damage. Yashiro et al. [17] conducted a case study of historical earthquake damage of the tunnels in Japan and outlined the failure mechanism of tunnel portals. Wang et al. [6] studied the effects of surrounding rock-structure interaction and hydrodynamic pressure on the seismic response of a hydraulic tunnel portal, and deduced two damage patterns for the tunnel portal. Tao et al. [4] analyzed the lateral and longitudinal deformation characteristics of a tunnel portal by shaking table model test, and concluded that the arch spandrel and arch springing suffer more damage than other positions. Apparently, those works can effectively rise our knowledge about the seismic behaviors of tunnel portals, but it should also note that most of them did not analyze the impacts of the incident angles of seismic waves. In fact, several statistics data of earthquake records suggests that actual earthquake motions propagate at rock medium obliquely because seismic waves does not experience enough refractions during their propagation [18-20], and under this scenario, the wave fields of tunnel portals are very complicated, and so as their seismic response. Currently, the works of assessment the effects of seismic waves incident angle on the seismic behaviors of tunnels are mainly concentrated on deep buried cylindrical traffic tunnels [21-23], while the effects on tunnel portals, especially for hydraulic tunnel portals having arch roof and vertical walls, have not been reported. In addition, hydraulic tunnels, compared to traffic tunnels, are additionally subjected to considerable hydrodynamic pressure during earthquakes, which is expected to increase the complexity of their seismic behaviors. The added mass method proposed by Housner [24] is currently the most common method used for simulating hydrodynamic pressure because it is simple and easy to implement [6,25], but it cannot properly reflect the fluid-solid coupling effects.

In light of this, the present work aims to develop a full three-dimensional (3D) numerical model, which could simulate the oblique input of seismic waves and the fluid-solid coupling effects, to assess the seismic stability of hydraulic tunnel portals under different incident angles of seismic waves.

2. Problem statement and 3D numerical model

2.1. Problem statement

This paper studies a numerical modelling technique for seismic stability assessment of a hydraulic tunnel portal under different incident angles. Three assumptions are proposed to simplify the analysis. Firstly, the tunnel portal is simplified as single free face slope with a slope angle of 45°. Secondly, the tunnel and surrounding rock are assumed to be tied together and the rock-tunnel interaction is not considered here. Nevertheless, since our target is the assessment of the impacts of incident angle of seismic waves, this assumption can be still accepted. Thirdly, the material properties of the tunnel and rock mass are regarded as uniform along the longitudinal axis of tunnel. Similar assumption can be also found in the works of Refs. [4, 20]. Based on the above assumptions, the problem we try to explore can be described as assessment of the seismic behaviors of a concrete lined hydraulic tunnel embedded in a rock slope of 45° slope angle (see Fig. 1).
2.2. Establishing of the FEM model

In this work, the finite element method (FEM) is employed to simulate the problem described in Section 2.1 because of its incomparable advantages in dealing with large-scale nonlinear problems. The simulation is conducted by an in-house dynamic finite element numerical simulation platform [26]. Fig. 1 offers the 3D finite element model used for simulation and the detail dimensions of the model can be found in Fig. 1. The Y-axis and Z-axis of the global coordinate system are defined by the tunnel longitudinal and vertical, while the X-axis by Right-hand rule. The entire mesh of the model is discretized with hexahedral elements, and a total of 166714 elements are meshed. According to the work of Ref. [27], the mesh size of the model should meet the accuracy requirements of dynamic calculation; thus, the maximum mesh size in this paper is chosen as 3 m. Fig. 2 shows the cross section of a hydraulic tunnel, which is made of reinforced concrete with a thickness of 0.6 m. The tunnel, which is embedded at a depth of 20 m, has a maximum height of 10 m and a width of 8m. In addition, the designed water depth is 7.50 m (see Fig. 2).

2.3. Analysis procedure

In general, the numeral simulation can be divided into three steps. The first step is to determine the initial stress field in equilibrium. The most appreciate method is to invert the geostress field based on measured data, but the measured data is not available under many scenario; thus the gravity field is employed as the initial geostress field by researchers. In addition, the lateral pressure coefficient $k$ is set as 1.4 here and the horizontal stress $\sigma_h$ can be given by $\sigma_h = k\sigma_v$. $\sigma_v$ is vertical stress and is induced by gravity varies linearly...
with depth. In the second step, static excavation calculation is performed and the stress field calculated in this step is adopted as the initial stress condition. In this process, the bottom boundary is fixed, the normal directions of lateral boundaries are fixed and the top boundary is free. Note that, to simply the calculate process, the excavation history is not considered. A similar simplification is also made in previous analyse [22]. In the third step, the dynamic response process of the tunnel portal is simulated. The mechanical behaviors of rock mass is modeled by Mohr-Coulomb criterion with a tension cut-off limit, and that of the concrete material by the plastic damage model [28]. In order to absorb the scattering waves from the calculation model to the infinite medium, the viscous spring artificial boundary presented by Liu et al. [29] is employed and applied in the bottom and lateral boundaries of the calculation model. The damping effects of the mediums are reflected by Rayleigh damping with a critical damp ratio of 5%. The acceleration time-history of the incident seismic waves, which is synthetized according to site conditions of the hydraulic tunnel, is depicted in Fig. 3, with a peak of 0.2 g. In order to evaluate the effects of incident direction of seismic waves on the seismic response of tunnel portal, four incident angles \( \theta \), which are 0°, 30°, 60° and 90°, are modeled. The Input of seismic waves is implemented by the method described in Section 3. Moreover, the fluid-solid coupling is considered by the method introduced in Section 4. The 3D FEM models proposed in Section 3 and Section 4 are implemented in the in-house dynamic numerical simulation platform [26] and are used for the analyses of this work.

**Table 1**

| Material         | Density (kg/m³) | Deformation modulus (GPa) | Poisson’s ratio | Friction angle (°) | Cohesion (MPa) | Tensile strength (MPa) |
|------------------|-----------------|---------------------------|-----------------|--------------------|----------------|------------------------|
| Rock mass        | 2.7             | 2.0                       | 0.30            | 35                 | 0.7            | 1.5                    |
| Concrete lining  | 2.5             | 27.5                      | 0.167           | 40                 | 1.75           | 1.25                   |
| Fluid            | 1.0             | 2.1                       | -               | -                  | -              | -                      |

Fig. 3 Time-history (left) and Fourier spectrum (right) of input acceleration.

3. **Input method of obliquely incident P waves**

3.1. **Seismic wave field at the artificial boundaries**
In the previous works of dynamic calculation, it is mostly assumed that seismic waves are incident vertically from the bottom boundaries of calculation models. This assumption can be accepted for underground structures in soils, but it is expected to contribute to an obvious deviation for that in rocks. According to the works of Refs. [6,20,22,30], oblique incident seismic wave is believed to increase the seismic response of underground structures. The main difference of the two scenarios lies on the fact that, the distances $\Delta L$ of different nodes on boundaries to the wave front are different when seismic waves incident obliquely, and thus seismic waves received by different nodes have phase difference. In addition, the energy of seismic waves continues to dissipate during their propagation in inelastic medium, which is manifested in the amplitude attenuation of seismic waves. Therefore, it is required to consider the two facts when deriving the wave field at the artificial boundary under oblique incidence of seismic waves.

Fig. 4 shows the propagation of oblique incidence of plane P waves. Assuming the displacement field of the wave front at zero time is $u_0(t)$, then the received displacement field $u_l(t)$ of node $l(x, y, z)$ can be given by

$$u_l(t) = \omega u_0(t - \Delta t)$$

(1)

Where $\omega$ is the coefficient of amplitude attenuation; $\Delta t$ is the delay time of the seismic waves propagating from the wave front to boundary node $l$, and can be given by the ratio of $\Delta L$ to wave velocity; $\Delta L$ is the distance between node $l$ to the wave front, and can be obtained according to the 3D spatial relationships of the nodes and the wave front (see Fig. 4); thus, there is

$$\Delta L = x_1 \sin \theta + z_1 \cos \theta \quad 0^\circ < \theta < 90^\circ$$

$$\Delta L = 0 \quad \theta = 0^\circ \text{ or } 90^\circ$$

(2)

$$\Delta t = \Delta L / c_p$$

(3)

Where $\theta$ represents the incident angle of P waves (see Fig. 4); $c_p$ is the velocity of P waves during their propagation. According to Eqs. (1) to (3), it is obvious that the key deriving the displacement $u_l(t)$ of boundary node $l$ is to obtain the amplitude attenuation coefficient $\omega$.

3.2. Attenuation coefficient of seismic waves during propagation

Due to the damping effects of inelastic mediums and the exist of geological discontinuities, the energy of seismic waves are proved to continually decay during their propagation [31] and the fluctuation amplitude is expected to decrease in this process. The attenuation mechanism of seismic waves is extremely complicated because it is believed to be affected by several parameters [32], e.g., seismic wave frequency, geological discontinuities, physical characteristics of material medium, propagation distance, etc. Obviously, it is
unrealizable to take all these parameters into account because their impact mechanism is still not entirely clear. Moreover, seismic wave frequency has been confirmed to have little effects on the attenuation of seismic waves propagating in dry homogeneous mediums [31,33], and geological discontinuities is not considered in this work; thus, only the physical characteristics of material medium and the propagation distance are considered here. It is well known that the longer of the propagation distance, the more attenuation of seismic waves. Then two assumptions can be introduced here to simply the problem: (1) the relationship between the attenuation coefficient \( \omega \) and propagation distance \( \Delta L \) is linear; (2) for every given case, the effects of the physical characteristics of material medium can be represented by an empirical constant. Based on the assumptions, the attenuation coefficient \( \omega \) can be given by

\[
\omega = \omega_0 \Delta L
\]  

Where \( \omega_0 \) is an empirical constant associated with the physical characteristics of material medium. A newly method was proposed by Zhang et al. [34] to derivate \( \omega_0 \), and the effects of elastic modulus and critical damping ratio of material on \( \omega_0 \) are considered in the method. A series of numerical tests conducted by Zhang et al. [34], and according to the test results, \( \omega_0 \) can be determined by Fig. 5. In the present study, the elastic modulus of rock mass is 2 GPa, and the critical damping ratio is 5%, so the empirical constant \( \omega_0 \) can be estimated as 0.0827 %. Combined Eqs.(1) to (4), the displacement field \( u(t) \) can be obtained accordingly.

![Fig.5 Relationships of \( \omega_0 \) with critical damping ratio under different elastic modulus [34].](image)

### 3.3. Equivalent nodal force

When the displacement field of boundary nodes is obtained, the next step is to input the seismic waves into the calculation model. Based on the viscoelastic artificial boundary theory, Liu et al. [35] realized wave input by transforming the incident waves into equivalent loads applied on the artificial boundaries. The equivalent load \( F_i \) can be determined by the following expression

\[
F_i = C_i \dot{u}_i + K_i u_i + \sigma_i A_i
\]  

Where \( \dot{u} \) is the velocity field of input seismic waves, and can be obtained by displacement \( u \) versus time; \( \sigma \) is the stress of input motion and can be determined by the generalized Hooke’s law; \( A_i \) is the equivalent area for the boundary node \( i \); subscript \( i \) (i = 1, 2, 3) represents three directions of node \( i \). \( C \) and \( K \) are the damping coefficient and elastic spring coefficient of viscoelastic artificial boundary, respectively. According to the viscoelastic artificial boundary proposed by Liu et al.[29], \( K \) and \( C \) can be expressed as follows

In the normal direction

\[
K_n = \frac{\alpha_n G}{r}, \quad C_n = \rho c_r
\]  

In the tangential direction

\[
K_t = \frac{\alpha_t G}{r}, \quad C_t = \rho c_t
\]
Where subscripts $N$ and $T$ represent normal and tangential directions of artificial boundaries; $r$ is the distance between load point with artificial boundaries; $G$ is the shear modulus; $\rho$ is the density of rock mass; $c_p$ and $c_s$ are the wave velocities of compression wave and shear wave; $\alpha_N$ and $\alpha_T$ are the modified coefficients and are suggested to be 1.33 and 0.67 [29], respectively. According to Eqs. (1) to (7), we can implement the input of obliquely incident P waves. Note that, the wave front is assumed parallel to the longitudinal axis of the tunnel in this work, so incident direction of the seismic waves is vertical to the longitudinal axis. The scenario in this assumption is a special case in actual engineering. Nevertheless, this assumption can be acceptable because compared with the scenario that the incident direction is not vertical to the longitudinal axis, the tunnel are proved to suffer larger seismic response when the incident direction is vertical to the longitudinal axis [22]; thus, the scenario studied in this work is an unfavorable case, which is also the scenario catches much concern in actual engineering.

4. Fluid-solid coupling analysis Method

During earthquakes, the large mass of water in hydraulic tunnels undoubtedly contributes to a further increase of the seismic response in hydraulic tunnels [6], and the seismic response hydraulic tunnels can be regarded as a complex coupling interaction process. In this work, an explicit method, which is of high efficiency for the large-scale nonlinear calculation problems, is employed to solve the motion equations of fluid and solid mediums, and then the coupling interaction conditions of the two mediums are introduced to establish the fluid-solid coupling analysis model.

4.1. Explicit iterative method for motion equations

It is assumed that the water in the hydraulic tunnel is a kind of constant and uniform fluid. After finite element discretization, the motion equations of solid and fluid mediums can be expressed as [36]

$$\begin{align*}
M_i \ddot{u}_i + C_i \dot{u}_i + K_i u_i = F_i + P_i \\
M_f \ddot{u}_f + C_f \dot{u}_f + K_f u_f = F_f.
\end{align*}$$

(8)  

(9)

Where subscripts $i$ and $f$ represent solid and fluid mediums, respectively; $K$, $C$ and $M$ are the stiffness matrixes, damping matrixes and mass matrixes, respectively; $u$, $\dot{u}$ and $\ddot{u}$ are displacement, velocity, acceleration of nodes; $F$ represents the contact force on the contact surface of fluid and solid mediums; $P$ is the known external forces.

Two basic methods are widely used to solve the motion equations: explicit algorithm and implicit algorithm. Explicit algorithm do not require iterative calculations, and thus it is of high stability and low time costing during calculation. However, it cannot be ignored the errors accumulation occurred in the process of explicit calculation. In this paper, the explicit algorithm, that is weighted residual approach, is employed because it constantly corrects the errors during the calculation process. The explicit integral format at time $n+1$ are thus can be given as

$$\begin{align*}
\Delta t \ddot{u}_i^{n+1} + \Delta t^2 \dddot{u}_i^{n+1} &= 2 \Delta t \dot{u}_i^n - u_i^n + \Delta t^2 \{ 2 \Delta t \Delta M_i^{-1} \Delta C_i \{ 2 \Delta t \dot{u}_i^n - u_i^n + \ddot{u}_i^n \} / 2 + \Delta t \Delta M_i^{-1} (F_i^n + P_i^n - K_i u_i^n) / 2 \\
\Delta t \dot{u}_i^{n+1} &= \Delta t \{ 2 \Delta t \Delta M_i^{-1} \Delta C_i \{ 2 \Delta t \dot{u}_i^n - u_i^n + \ddot{u}_i^n \} / 2 + \Delta t \Delta M_i^{-1} (F_i^n + P_i^n - K_i u_i^n) / 2 \\
F_i^{n+1} &= 2 \Delta t \Delta M_i^{-1} \{ u_i^{n+1} - (u_i^n - u_i^n) / \Delta t \} / \Delta t + C_i \{ u_i^{n+1} - u_i^n \} / \Delta t + K_i u_i^{n+1} - P_i^{n+1} \\
\Delta t \ddot{u}_f^{n+1} + \Delta t^2 \dddot{u}_f^{n+1} &= 2 \Delta t \dot{u}_f^n - u_f^n + \Delta t^2 \{ 2 \Delta t \Delta M_f^{-1} \Delta C_f \{ 2 \Delta t \dot{u}_f^n - u_f^n + \ddot{u}_f^n \} / 2 + \Delta t \Delta M_f^{-1} (F_f^n - K_f u_f^n) / 2 \\
\Delta t \dot{u}_f^{n+1} &= \Delta t \{ 2 \Delta t \Delta M_f^{-1} \Delta C_f \{ 2 \Delta t \dot{u}_f^n - u_f^n + \ddot{u}_f^n \} / 2 + \Delta t \Delta M_f^{-1} (F_f^n - K_f u_f^n) / 2 \\
F_f^{n+1} &= 2 \Delta t \Delta M_f^{-1} \{ u_f^{n+1} - (u_f^n - u_f^n) / \Delta t \} / \Delta t + C_f \{ u_f^{n+1} - u_f^n \} / \Delta t + K_f u_f^{n+1}
\end{align*}$$

(10)  

(11)

Where $\Delta t$ is the time step.
According to the introduction in Section 2.3, the stress field obtained by static excavation calculation is adopted as the initial stress condition, and the displacement and velocity at $t = 0$ are assumed as zero; thus, here are

$$\begin{align*}
\ddot{u}^i &= \ddot{u}^s = 0 \quad (12) \\
\dot{u}_1^i &= M_1^i (F^i + P^i) \quad (13) \\
\dot{u}_2^i &= M_2^i F^i \quad (14) \\
\Delta^2 u^j &= \Delta^2 u^j / 2 \quad (15)
\end{align*}$$

From Eqs. (10) to (15), it is clear that the last unknown for the solution of the integral equations (10) and (12) is the contact force $F$, so it is a key step to determine the contact force $F$.

### 4.2. Coupling interaction conditions

The fluid-solid coupling of the water and the tunnel during earthquakes can be seen as a process of dynamic interaction, so the idea of dynamic contact can be used to deal with the fluid-solid coupling problem. The contact characteristics of the two is that the normal interaction is little and can be ignored, and the coupling effects are mainly dominated by the tangential interaction. According to the principle of interaction, the normal stress and normal displacement of the fluid and solid symmetrical on the two sides of their contact face can be regarded as be continuous; thus, here is

$$\begin{align*}
F_{11} &= F_{12} = F_{21} = F_{22} = 0 \quad (16) \\
F_{13} + F_{23} &= 0 \quad (17) \\
\dot{u}_{13} &= \dot{u}_{23} \quad (18) \\
\ddot{u}_{13} &= \ddot{u}_{23} \quad (19)
\end{align*}$$

Where subscript 3 refers to the normal direction of the contact face; subscripts 1 and 2 represent two tangential interaction.

Substituting the interaction conditions and the initial stress conditions, i.e., Eqs. (16) to (19) and Eqs. (12) to (15), into Eqs. (10) and (11), and here is

$$\begin{align*}
\Delta^2 u_{1j}^i &= \ddot{u}_{1j} + \Delta \ddot{u}_{1j} - \frac{\Delta t}{2M(M + M_j)} \left[ 2C_j \left( 2\Delta \dot{u}_j^i - \dot{u}_j^i + \dot{u}_j^{i-1} \right) + C_j \left( 2\Delta \dot{u}_j^i - \dot{u}_j^i + \dot{u}_j^{i-1} \right) \right] + \frac{\Delta t}{2M(M + M_j)} \left( K_{2j} \dot{u}_j^j + K_{2j} \dot{u}_j^j \right) \quad (20) \\
\Delta^2 u_{2j}^i &= \ddot{u}_{2j} + \Delta \ddot{u}_{2j} - \frac{\Delta t}{2M(M + M_j)} \left[ 2C_j \left( 2\Delta \dot{u}_j^i - \dot{u}_j^i + \dot{u}_j^{i-1} \right) + C_j \left( 2\Delta \dot{u}_j^i - \dot{u}_j^i + \dot{u}_j^{i-1} \right) \right] + \frac{\Delta t}{2M(M + M_j)} \left( K_{2j} \dot{u}_j^j + K_{2j} \dot{u}_j^j \right) \quad (21)
\end{align*}$$

Substituting the displacement $\ddot{u}$ and velocity $\dot{u}$ into Eqs.(10) and (11), and then the contact force $F$ can be determined.

### 5. Results and discussions

In this section, the proposed simulation methods in Section 3 and Section 4 are used to evaluate the seismic stability of the tunnel portal under different incident angles. To analysis the effects of incident angles of P waves, four incident angles $\theta$, i.e., 0°, 30°, 60°, 90°, are modeled here. Moreover, two comparative studies, one consider hydrodynamic pressure and the other do not, are performed to study the impacts of hydrodynamic pressure. Additionally, three sections (S1, S2, S3) and one monitoring point (point A) (see Fig. 6) are chosen to illustrate the results.
5.1. Stress response under different incident angles

Fig. 7 Maximum principle stresses under different incident angles. (a): time histories of the stresses of point A at Section S1; (b) Peaks of the stresses of point A at the three sections.

The maximum principle stress is employed here to assess the tension state of the tunnel. Fig. 7 (a) and (b) display the time-histories of the maximum principle stress of tunnel crown (point A on Section S1) and the peaks of the stresses on the three Sections, respectively. It can be found in Fig. 7 (a) that the tunnel crown is in tension state after the static excavation (t = 0 s) and the tensile stress is approximately 0.5 MPa. As time increases, the stresses suffer a violent fluctuation in the period of 5s – 15s, and the values of stress curves range from -0.66 MPa to 2.37 MPa during this period. It is interesting to note that the primary difference of stress curves under different incident angles lies in the stress amplitude, which is evidently varies with the incident angles. In addition, the amplitude of maximum principle stress is what we are extremely concerned about because concrete materials are widely believed to be of weak tensile strength, and thus it is commonly adopted as an indicator of damage. Fig. 7(b) plots the stress amplitudes of the tunnel crowns on the three sections under different incident angles. It can be found from Fig. 7(b) that section S1 suffers a larger stress amplitude than the other two sections, and all the stress amplitudes of section S1 under the four incident angles exceed the tensile strength (1.25 MPa, see Table 1) of the concrete material studied in this paper, suggesting a tension damage on tunnel crown on Section S1. In addition, the stress amplitudes under the scenarios of 30° and 60° incident angles are much larger than that of the scenarios of 0° and 90° incident angles, which indicates that oblique incident P waves are likely to contribute a larger stress response of the tunnel. According to the results plotted in Fig. 7, we can conclude that the stress induced by obliquely incident P waves is larger than that induced by horizontal and vertical incident P waves. Moreover, the closer to the tunnel portal, the larger of the stress amplitude. Similar conclusions can be also found in Refs. [6, 22].
5.2. Displacement response under different incident angles

Fig. 8 Displacement of point A under different incident angles. (a): time histories of the displacements of point A at Section S1; (b) Peaks of the displacements of point A at the three sections.

Displacement response is another indicator to evaluate the seismic stability of the tunnel portal. Fig. 8(a) displays the time-histories of the tunnel crown on Section S1 under different incident angles. It is clear that the four fluctuation curves are highly similar to each other, and the peaks of these curves nearly occurs at the same time. Additionally, the major difference of displacement curves under the four incident angles is that the displacement amplitude varies with the incident angles, and the displacement amplitude has a maximum when the incident angle is 30°. These results indicate that the incident direction of P waves has significant effects on the displacement amplitude but little effects on the fluctuation characteristics of the displacement curves. Fig. 8(b) shows the peaks of the displacement curves of the tunnel crowns on the three sections under different incident angles. Two characteristics shown in Fig. 8(b) impressed us. One is that the displacement peak of the tunnel decreases with the increasing of the distance to the tunnel portal. The other is that the displacement peak increases first, and then decreases with the increasing of the incident angle, and it reaches the maximum value at the case of 30° incident angle. This result indicates that oblique incident seismic waves contribute a larger dynamic displacement response. From the results in Fig. 8, it can be concluded that the displacement response induced by oblique incident P waves is much larger than that by horizontal and vertical incident seismic waves. In addition, the closer to the tunnel portal, the larger of the stress response.

5.3. Seismic damage under different incident angles

The damage coefficient of the tunnel is adopted in this section to quantitatively show how much seismic damage is induced by incident P waves and where suffers severe damage. The damage coefficient $D$, according to the introduction in Section 2.3, can be given by

$$D = 1 - (1 - d_c)(1 - d_t)$$

$$d_c = 1 - \left(1 - \frac{1}{\varepsilon_s} \right) \frac{A}{\exp[B_s(\varepsilon_s - \varepsilon_s')]}$$

$$d_t = 1 - \left(1 - \frac{1}{\varepsilon_t} \right) \frac{A}{\exp[B_t(\varepsilon_t - \varepsilon_t')]}$$

(22)

Where $d_c$ and $d_t$ are compressive damage variable and tension damage variable, respectively; $s$ is the stiffness restitution coefficient; $A_s$ and $B_s$ are shear damage parameters, $A_t$ and $B_t$ are tensile damage parameters, and they can be obtained according to experimental results; $\varepsilon_s'$ and $\varepsilon_t'$ are thresholds of strain for shear and tensile damage; $\varepsilon_s'$ is equivalent plastic strain.

The damage evolution process of the tunnel under different incident angles is presented in Fig. 9. As shown in Fig. 9, the tunnel damage increases with time. The damage areas are mainly distributed on the
tunnel within 40m from the tunnel portal when time is 5 s. With time increasing, the damage zone continues to expand along the tunnel axis (Y-axis) and the damage degree is growing in this process. At the end of the dynamic simulation (t = 20 s), the tunnel is almost completely covered by damage zone, and the most positions of the tunnel portal suffers severe damage \((D > 0.7)\). The results indicates that the tunnel damage in the earthquake is a continuous accumulation process, and tunnel portal is where damage occurs first and suffers severe damage. In addition, the comparison of the results under different incident angles shows that the damage areas and damage degree of the tunnel under 30° (Fig. 9b) and 60° (Fig. 9c) incident angles are more severe than that under 0° (Fig. 9a) and 90° (Fig. 9d) incident angles, and the tunnel suffers the most severe damage in the case of 30° incident angle. These results suggest that the oblique incident seismic waves are likely to lead to a more severe damage.

Fig.9 Damage coefficient distribution of the tunnel under different incident angles. (a): 0° incident angle; (b) 30° incident angle; (c) 60° incident angle; (d) 90° incident angle.

5.4. Impact of hydrodynamic pressure on the seismic response

A new parameter named global damage \(D_g\) is introduced in the section to display the effects of hydrodynamic pressure on mechanical behaviors of the tunnel portal. \(D_g\) can be given by

\[
D_g = \frac{1}{n} \sum_{i=1}^{n} \frac{V_i^d}{V}
\]

(23)

Where \(n\) is the number of damaged elements; \(V_i^d\) is the volume of damaged element; \(V\) is the
volume of all elements.

Fig. 10 plots the time histories of $D_t$ under two cases. In case 1, hydrodynamic pressure is modeled by the method presented in Section 4; while in case 2, hydrodynamic pressure is not considered and the water in the hydraulic tunnel is modeled by the added mass method [37]. It is obvious that $D_t$ experiences a continuous increase in the period of 5 s to 15 s under the two cases. This result is consistent with the result of stress response shown in Fig. 7, which also suffers a violent fluctuation during this period. The main difference of the two cases lies on the fact that the global damage $D_t$ in case 1 is larger than that of case 2, which means hydrodynamic pressure is likely to contribute to a more severe damage.

Fig. 10 Time histories of global damage under different cases. Case 1: with hydrodynamic pressure; Case 2: without hydrodynamic pressure

6. Summary and conclusions

By deriving the distance of the nodes to wave front and the attenuation coefficient, an input method of obliquely incident P waves, which can reflect the phase difference and the damping effect of inelastic medium during the seismic wave propagation, is proposed. Combined with the explicit fluid-solid coupling analysis model, a dynamic response analysis method for hydraulic tunnels is established. The method is employed to assess the effects of the incident angles of P waves on the seismic stability of a hydraulic tunnel, and the major conclusions can be drawn as follows:

The stress amplitudes of the tunnel crown under the cases of 30° and 60° incident angles are much larger than that of the cases of 0° and 90° incident angles;

The displacement amplitude of the tunnel crown increases first, and then decreases with the increasing of the incident angle, and has a maximum when the incident angle is 30°;

The closer to tunnel portal, the larger seismic response of the tunnel, and tunnel portal is where damage occurs first and suffers severe damage;

The damage areas and damage degree of the tunnel under oblique incident seismic waves are more severe than that under horizontal and vertical incident seismic waves.

Compared with the case that hydrodynamic pressure is not considered, the damage volume of the ground-tunnel system is larger when hydrodynamic pressure is considered.

Additionally, some limits of our present work should be mentioned. Only the impacts of P waves on seismic response of the tunnel portal are considered here, while the effects of S waves and R waves are not
studied. Moreover, it is assumed that the tunnel and surrounding rock are tied together, and the dynamic ground-tunnel interaction are not considered. In fact, the ground-tunnel interaction is likely to have significant effects on the mechanical behaviors of the ground-tunnel system. A further study regarding these issues is in the process.

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

8. Data statement
Data Is Available Upon Request.

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