Analysis of seismic damage process of high concrete dam-foundation system

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Abstract: In the anti-seismic analysis of concrete high dams, only the damage cracking of the dam concrete is usually considered, and the nonlinear characteristics of the dam foundation rock during the earthquake are not considered. Taking the Koyyna gravity dam in India and Shapai arch dam in China as examples, this paper adopts the nonlinear time-history analysis method and takes into account the comprehensive influence of the radiation damping of infinite foundation, the hydrodynamic interaction between dam and reservoir, the nonlinear dynamic contact between various joints in the dam body and foundation, and the damage of dam concrete and foundation rock mass in the damage evolution process, analyzes damage evolution process of dam body-foundation system under earthquake action and the influence of foundation rock mass damage on seismic failure mode of concrete dam-foundation system, and compares the numerical simulation results with the actual earthquake situation. The calculation results show that after considering the damage of the dam foundation rock mass, the rock mass of the dam heel firstly appears to be damaged under the action of the earthquake and gradually expands into the interior of the foundation. The dam heel and the foundation surface are not damaged. The numerical simulation results are more in line with the actual situation of earthquake.

1. Introduction

In the anti-seismic design and numerical simulation of concrete dams, the damage cracking of the dam foundation rock mass is not considered, and the dam site often has large tensile stress. The dam may cause damage from the dam-foundation interface. Therefore, the concrete dam is designed to avoid large tensile stresses in the dam. However, at present, several high concrete dams have been built in China, such as the Three Gorges gravity dam, Xiluodu arch dam, Xiaowan arch dam, Jinping first-class arch dam, etc., which dam stresses monitored after impoundment are compressive stress, the calculation result of dam heel stress is inconsistent with the monitoring value. In addition, China's Shapai arch dam has withstood the strong earthquake test beyond the design earthquake in the 5.12 Wenchuan Earthquake, and there is no damage cracking in the dam heel area [1]. In 1971 and 1994, the Pacoima arch dam in the United States suffered strong earthquakes magnitude 6.6 and 6.8. In 1971, the transverse joint between the dam and the left bank gravity pier was opened, but the dam itself did
not damage [2]. In 1967, the Koyna gravity dam in India experienced horizontal cracks in the dam slopes of the downstream dams of several dam sections after the 6.3 magnitude earthquake, but the dam and the foundation surface did not crack [3–5]. The dam stress monitoring data and existing earthquake cases show that the existing anti-seismic design and numerical simulation methods are difficult to reflect the actual situation.

In fact, there are some randomly distributed fissures in the rock foundation of the arch dam. Due to the poor tensile strength of the rock mass fissures, the rock mass fissures may be repeatedly tensioned under the action of the earthquake, thus affecting the seismic response of the arch dam. This kind of viewpoint can be confirmed from the existing earthquake cases. For example, after the 1971 earthquake, the Pakoyima arch dam was used for the core test of the dam body for pressure test and intra-hole photography. The results did not find a relative displacement between the body and the bedrock. Further investigation revealed that new fractures and re-opening of the original fractures were found in the foundation rock mass [6]. Therefore, in the seismic safety evaluation of arch dams, the dynamic nonlinear behavior of the rock mass of the dam foundation should be considered. Dam concrete and dam foundation rock mass act as quasi-brittle materials, and when the stress exceeds its strength value, it will cause damage cracking and exhibit strain softening characteristics. Therefore, it is reasonable to use the dynamic damage constitutive model to simulate the material nonlinearity of dam concrete and dam foundation rock under earthquake [7-11].

In addition, in the analysis of anti-seismic safety of arch dams, the radiation damping effect of infinite foundation and the nonlinear effects of dynamic contact of various types of joints should be considered. The anti-seismic stability check of the potential sliding block of the arch dam abutment and the anti-seismic strength check of the dam should be unified, the dynamic coupling between the dam and the abutment rock mass should be considered, it can reflect the change of the rock mass sliding mode with time, which are caused by the change of the arch thrust during the earthquake, and the dynamic amplification effect of the potential slider.

Therefore, in order to carry out the seismic response analysis of concrete high dams more objectively, this paper takes the dam body and the near-field foundation as a system, considering the radiation damping effect of the infinite foundation, the hydrodynamic interaction between dam and reservoir, the various types of seams in the dam and the dam foundation. The nonlinear dynamic time history analysis method is used to analyze the damage process of the Koyna gravity dam in India and the Shapai arch dam in China under strong earthquakes. The influence of foundation rock damage on the seismic failure mode of concrete dam-foundation system.

2. Analysis theory and method

2.1 Infinite foundation boundary conditions and ground motion input

In the seismic response analysis of concrete high dams, the energy dissipation to the remote foundation cannot be ignored. In this paper, the equivalent uniform viscoelastic boundary element is used to simulate the radiation damping effect of the infinite foundation, that is, the continuous solid element is used to replace the viscoelastic artificial boundary composed of discrete spring-damper elements. The shear modulus \( \tilde{G} \), elastic modulus \( \tilde{E} \), and damping scale factor \( \tilde{\eta} \) of the equivalent viscoelastic boundary element can be calculated[12-14]

\[
\tilde{G} = hK_r = a_r h \frac{G}{r} \tag{1}
\]

\[
\tilde{E} = \frac{(1 + \tilde{\nu})(1 - 2\tilde{\nu})}{(1 - \tilde{\nu})} hK_N = a_N \frac{G (1 + \tilde{\nu})(1 - 2\tilde{\nu})}{(1 - \tilde{\nu})} \tag{2}
\]

\[
\tilde{\eta} = \frac{\rho r}{3G} \left( \frac{c_s}{a_t} + \frac{c_p}{a_N} \right) \tag{3}
\]

Where: \( K_N, K_r \) are the normal stiffness and tangential stiffness of the viscoelastic artificial...
boundary spring; \( r \) is the distance from the scattered wave source to the artificial boundary; \( \tilde{v} \) is the equivalent Poisson's ratio of the equivalent uniform viscoelastic boundary element; \( G \) is the shear modulus of the medium; \( h \) is the thickness of the equivalent boundary element; \( a_r \) and \( a_n \) are the tangential and normal viscoelastic artificial boundary correction coefficients respectively; \( \rho \) is the mass density of the medium; \( c_s \) and \( c_p \) are wave speed of S wave and P wave, respectively.

When seismic input is made, the total wave field at the bottom boundary of the finite element model can be decomposed into the incident field and the scattered field, and the total wave field at the side boundary can be decomposed into a free field and a scattered field. According to the continuous condition of displacement and the condition of mechanical equilibrium, the equations of motion at any point \( b \) on the bottom boundary and the side boundary can be expressed as

\[
\dot{m}_b \ddot{u}_b^m(t) + C_b \dot{u}_b^m(t) + K_b u_b^m(t) = F_b^r(t) + F_b^s(t)
\]

\[
\dot{m}_b \ddot{u}_b^m(t) + C_b \dot{u}_b^m(t) + K_b u_b^m(t) = F_b^f(t) + F_b^s(t)
\]

Where: \( K_b, C_b \) are the spring stiffness and damping of the artificial boundary node, and \( F_b^r(t), F_b^f(t), \) and \( F_b^s(t) \) are the equivalent node loads to be applied to the artificial boundary node by the simulated boundary incident field, free field, and scattering field, respectively.

### 2.2 Damage model

In this paper, the dynamic damage model is used to simulate the nonlinear mechanical behavior of dam concrete and dam foundation rock mass. In order to describe the damage evolution of concrete under multiaxial stress conditions, this paper uses the yield criterion proposed by Lubliner [15] and modified by Lee and Fenves [16] as the damage criterion of concrete, in the effective stress space, the damage surface mathematical expression is

\[
F = \frac{1}{1 - \alpha} \left( \bar{\sigma}_1 + \sqrt{3J_2} + \beta \langle \tilde{\sigma}_{\text{max}} \rangle - \sigma_c \right)
\]

Where: \( \bar{\sigma}_1 \) is the first principal invariant of the effective stress; \( J_2 \) is the effective stress bias second non-main variable; \( \tilde{\sigma}_{\text{max}} \) is the maximum algebraic eigenvalue of the effective stress; \( \langle x \rangle \) means \( \langle x \rangle = (|x| + x) / 2 \); \( \alpha \) is a dimensionless material constant, its value can be determined by the expression \( \alpha = \frac{f_{b0} - f_{c0}}{2f_{b0} - f_{c0}} \), where \( f_{b0}, f_{c0} \) is the compressive strength of the biaxial compression and uniaxial compression, respectively; \( \beta \) can reflect the evolution of the damage surface, and its value can be determined by the expression \( \beta = \frac{\sigma_i}{\sigma_t} (1 - \alpha) - (1 + \alpha) \), \( \sigma_i \) and \( \sigma_t \) are tensile strength and compressive strength, respectively.

Due to the different evolution laws of the tensile and compressive stress of concrete, the reverse compression of the material after the tensile damage will lead to the recovery of its stiffness. Therefore, the introduction of a weighting factor \( r(\tilde{\sigma}) \) that considers the interaction between tensile and compressive stresses in complex stress states reflects the “unilateral effect” of elastic mode recovery. The weighting factor is defined as

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

Damage variables under multiaxial stress conditions can be expressed as
\[ D = 1 - (1 - D_t)(1 - r(\sigma)D_c) \]  
(8)

Where: \( D_t \) and \( D_c \) are the tensile and compressive damage variables under uniaxial stress state.

Since the tensile strength of concrete and rock mass is much lower than its compressive strength, the damage of the material is mainly caused by stretching. In this paper, only the tensile damage of concrete and rock mass is considered in the seismic response analysis of concrete dam.

2.3 Dynamic contact model
Under the action of static load, the transverse joint between the dam sections of the arch dam is compacted, and the dam body is relatively intact. However, under the action of strong earthquakes, due to the low tensile strength between the transverse joints of the arch dam, the transverse joints may be pulled apart, and the complex movement process of reciprocating opening and closing and slipping is presented, as a result, the integrity of the dam decreased and caused the adjustment and redistribution of the stress of the dam. In addition, various geological tectonic planes in the dam foundation will reciprocate open and close under the action of strong earthquakes, showing obvious nonlinear effects of dynamic contact. Therefore, this paper uses the constraint function contact algorithm [17~19] to simulate the transverse joint of the arch dam and the potential slider structure surface of the dam foundation.

3. Seismic damage analysis of Koyna gravity dam-foundation

3.1 Calculate models, parameters, and seismic loads
In this paper, a typical non-overflow dam section of the Indian Koyna gravity dam subjected to actual earthquake damage is selected. The section size of the dam section is shown in Figure 1. The finite element dispersion of the dam is carried out by a planar 4-node unit, and the Koyna gravity dam-foundation is established. The finite element model of the system has a total of 32,608 units and a total of 66,067 nodes, as shown in Figure 2.

The parameters of the dam concrete material used in the calculation are [6]: concrete elastic modulus \( E_0 = 30\text{GPa} \), Poisson's ratio \( \mu = 0.2 \), density \( \rho = 2630\text{kg/m}^3 \), tensile strength \( f_{t0} = 2.9\text{MPa} \), compressive strength \( f_c = -24.1\text{MPa} \), and fracture energy \( G_f = 200\text{N/m} \). The parameters of the foundation material used in the calculation are [20]: concrete elastic modulus \( E_0 = 20\text{GPa} \), Poisson's ratio \( \mu = 0.2 \), density \( \rho = 2700\text{kg/m}^3 \), tensile strength \( f_{t0} = 1.28\text{MPa} \), compressive strength \( f_c = -10\text{MPa} \), and fracture energy \( G_f = 88.4\text{N/m} \).

Computational loads include dam body weight, hydrostatic pressure, hydrodynamic pressure, and seismic loads. In the numerical analysis, the hydrodynamic pressure is simulated by the Westgaard
additional mass method, which does not account for the water compressibility of the reservoir. The damping is Rayleigh damping with a damping ratio of 0.05. The measured forward and vertical seismic waves are input respectively. The time history of the seismic wave is shown in Figure 3. The peak acceleration of the river is 0.53g, and the vertical peak acceleration is 0.35g. The calculation takes 0.01s each step and the total take 10s.

3.2 Analysis of dam body-foundation damage process

Figure 4 shows the damage distribution cloud map of the dam-foundation at different times. It can be seen that at the earthquake of 2.77s, the foundation rock mass of the dam site first appeared to be damaged, and gradually extended to the deep part of the dam foundation as the earthquake continued. At the 4.15s of the earthquake, the upstream surface of the dam neck was damaged, and the damage at the downstream slope of the dam was further extended to the upstream side. With the development of the earthquake time course, the upper and lower damages of the dam neck continue to expand into the interior of the dam, and the damage area of the dam neck is expanding. At the end of the earthquake, the damage of the neck of the dam basically runs along the upstream and downstream surfaces, and the foundation near the dam has a certain depth of damage, but the joint between the dam and the foundation does not cause damage, and the dam seepage prevention curtain wasn’t damaged, which is in line with the actual earthquake.
4. Seismic damage analysis of Shapai arch dam-foundation

4.1 Calculate models, parameters, and seismic loads

The dam crest elevation of Shapai arch dam is 1867.5m and the maximum dam height is 130m. The normal water level is 1866m, the silt elevation is 1796m, and the silt capacity is 500kg/m$^3$. The three-dimensional finite element discretization of the Shapai arch dam and the dam foundation is carried out. The total number of units is 20,911 and the total number of nodes is 25,395. The coordinate system is taken as: the direction of the river is toward the right bank; the direction is toward the river, pointing to the upstream; the direction is the vertical direction. The finite element meshes of the dam body and the sliding blocks of the left and right bank dams and the dam are shown in Figures 6 and 7, respectively.

The parameters of the dam concrete material used in the calculation are [21]: concrete elastic modulus $E_0 = 18$GPa, Poisson's ratio $\mu = 0.167$, density $\rho = 2400$kg / m$^3$, tensile strength $f_{t_0} = 5.3$MPa, compressive strength $f_c = -25$MPa, and fracture energy $G_f = 350$N / m. The parameters of the foundation materials used in the calculation are: concrete elastic modulus $E_0 = 11$GPa, Poisson's ratio $\mu = 0.25$, density $\rho = 2700$kg / m$^3$, tensile strength $f_{t_0} = 1.2$MPa, compressive strength $f_c = -25$MPa, and fracture energy $G_f = 200$N / m. The material parameters and strength indexes of the dam foundation rock mass are shown in Table 1.

According to the combined conditions of the abutment slip boundary provided by the geological section in the technical application stage, the left abutment combined with lateral cut surface N50°/ SE < 75° E, bottom sliding surface N30° E/SE < 30°, the downstream cut surface N40° W/SW < 70° to form a slide control block; the right abutment combined with lateral cut surface N30° E/NW < 60°, bottom sliding surface N45° W/NE < 30° combinations to form a control sliding block. The comprehensive strength indexes of the potential slip surfaces of the left and right abutments are shown in Table 1.
Table 1 Integrated strength indices of the potential slide surface

| Boundary structure surface               | Shear strength $f''$ | Shear strength $c'(\text{MPa})$ | Shear strength $f$ |
|-----------------------------------------|----------------------|----------------------------------|-------------------|
| Lateral cutting surface at left bank    | 0.99                 | 0.86                             | 0.74              |
| Bottom slip surface at left bank        | 1.11                 | 1.10                             | 0.80              |
| Lateral cutting surface at right bank   | 1.00                 | 0.88                             | 0.75              |
| Bottom slip surface at right bank       | 1.12                 | 1.13                             | 0.82              |

The calculated load combination is: normal water storage level + silt + dam body weight + design temperature drop + seismic load. In the numerical analysis, the hydrodynamic pressure is simulated by the Westgaard additional mass method, which does not account for the water compressibility of the reservoir. The damping is Rayleigh damping with a damping ratio of 0.05. Seismic response analysis was carried out using Maoxian wave, and the transverse river direction, the river direction and the vertical seismic wave were input respectively. As shown in Figure 8, the vertical acceleration was $2/3$ of the horizontal direction and the peak acceleration of the horizontal river was 2.57g. The peak acceleration is 2.57 g and the vertical peak acceleration is 1.71 g. The calculation takes 0.02 s for each step and a total of 65 s.

![Acceleration time histories of earthquake (Maoxian)](image)

(a) the transversal direction  (b) the longitudinal direction  (c) the vertical direction

Fig.8 Acceleration time histories of earthquake (Maoxian)

4.2 Analysis of dam body-foundation damage process

Figure 9 is a cloud diagram of the damage distribution at different moments of the arch crown beam of the Shapai arch dam. It can be seen that the foundation of the arch dam is first damaged at 3.02 s. As the earthquake continues, the damage gradually expands deep into the dam foundation. At the end of the earthquake, the rock mass within a certain depth of the dam base is damaged, but there is no damage at the interface between the dam itself and the dam foundation.
Figure 10 is a cloud diagram of the dam-foundation damage distribution at different elevations at the end of the earthquake. Figure 11 is a cloud diagram of the damage distribution of the dam at the end of the earthquake. It can be seen that at the end of the earthquake, the damage mainly occurred in the foundation near the base surface of the upstream dam surface, and the damage extended to both sides. The damage of the 1750m elevation was deeper, and the concrete dam did not damage, which is consistent with the actual earthquake.

Fig.9 Damage distribution of arch crown section at different moments

Fig.10 Damage distribution of dam-foundation at different altitudes at the end of earthquake
5. Conclusion
In this paper, the dynamic damage constitutive model is used to simulate the nonlinear mechanical behavior of concrete and dam foundation rock under earthquake. The equivalent uniform viscoelastic boundary element is used to simulate the radiation damping effect of infinite foundation, and the constraint function method is used to simulate the transverse joint and abutment of arch dam. The dynamic contact nonlinear effect of the sliding rock block surface is possible. The nonlinear time history analysis method is used. Taking the Koyna gravity dam in India and the Shapai arch dam in China as examples, the nonlinear seismic response analysis of the dam-foundation is carried out, and the dam is analyzed. The damage evolution process of the dam body-dam foundation system under earthquake action is studied. The influence of foundation rock damage on the seismic failure mode of concrete dam body-dam foundation system is studied, and the numerical simulation results are compared with the actual earthquake situation. The calculation results show that after considering the damage of the dam foundation rock mass, the rock mass of the dam site firstly appears to be damaged under the action of the earthquake and gradually expands into the interior of the foundation. The dam body and the dam foundation surface are not damaged. The numerical simulation results are more in line with the actual earthquake situation.

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