Reservoir water effects on earthquake performance evaluation of Torul Concrete-Faced Rockfill Dam

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Abstract: This study presents earthquake performance analysis of the Torul Concrete-Faced Rockfill (CFR) Dam with two-dimensional dam-soil and dam-soil-reservoir finite element models. The Lagrangian approach was used with fluid elements to model impounded water. The interface elements were used to simulate the slippage between the concrete face slab and the rockfill. The horizontal component of the 1992 Erzincan earthquake, with a peak ground acceleration of 0.515 g, was considered in time-history analysis. The Drucker-Prager model was preferred in nonlinear analysis of the concrete slab, rockfill and foundation soil. The maximum principal stresses and the maximum displacements in two opposite directions were compared by the height of the concrete slab according to linear time-history analysis to reveal the effect of reservoir water. The changes of critical displacements and principal stresses with time are also shown in this paper. According to linear and nonlinear time-history analysis, the effect of the reservoir water on the earthquake performance of the Torul CFR Dam was investigated and the possible damage situation was examined. The results show that the hydrodynamic pressure of reservoir water leads to an increase in the maximum displacements and principal stresses of the dam and reduces the earthquake performance of the dam. Although the linear time-history analysis demonstrates that the earthquake causes a momentous damage to the concrete slab of the Torul CFR Dam, the nonlinear time-history analysis shows that no evident damage occurs in either reservoir case.

Key words: earthquake performance evaluation; concrete-faced rockfill dam; dam-soil-reservoir interaction; Drucker-Prager model; interface element; Lagrangian approach; nonlinear time-history analysis

1 Introduction

Concrete-faced rockfill (CFR) dams are known as a low-cost alternative to clayey core rockfill (CCR) dams. Clayey core material can be obtained from the surroundings but it cannot be provided in the project area at all times (Uddin 1999). CFR dams have been built in several parts of the world in recent years (Seed et al. 1985). The popularity of CFR dams will probably increase in the future along with worldwide experience of their long-term performance.

The response of the concrete slab to earthquakes is a basic concern of a CFR dam (Guros et al. 1984; Seed et al. 1985; Arrau et al. 1985; Bureau et al. 1985; Priscu et al. 1985; Sherard...
and Cooke 1987; Gazetas and Dakoulas 1992; Uddin and Gazetas 1995; Uddin 1999). In the course of strong earthquakes, a concrete slab may be crushed and is likely to be cracked. The earthquake-induced failure may cause major cracks in the concrete slab and, as a result, the safety factor of the dam may decrease due to seepage failure (Cooke 1993). The concrete slab covers the upstream face of the rockfill, and if it cracks, water will penetrate the dam and may reduce its stability (Szostak-Chrzanowski and Massiéra 2006). Yoon et al. (2002) have showed that leakage-induced failure is a serious issue for concrete dams. Bureau et al. (1985) have presented a study relevant to the seismic performance of rockfill dams and especially to probable failure modes of CFR dams. Several researchers have also investigated dynamic failure of CFR dams (Han et al. 1988; Kong and Liu 2002).

The earthquake behavior of the concrete slab on a CFR dam mostly pertains to the interface between the concrete and rockfill. Different researchers have focused on the isolated discrete joints for nonlinear analysis (Goodman et al. 1968; Zienkiewicz et al. 1970; Mahtab and Goodman 1970; Ghaboussi et al. 1973). Interface elements allow separate movements between the solids and transfer shear deformations across the interfaces. Tzamtzis and Nath (1992) utilized three-dimensional interface elements in nonlinear static and dynamic finite element analysis of discontinuous systems. In addition, interface elements allowing sliding between soil and reinforcement according to the Mohr-Coulomb strength criterion were used to examine the interface behavior in reinforced embankments on soft ground (Hird and Kwok 1989).

Investigation of the effects of hydrodynamic pressures on the dynamic response of dams began in the 1930s (Westergaard 1933; Zangar and Haefei 1952; Zienkiewicz and Nath 1963). Many researchers have studied the dynamic response of dam-reservoir systems using the Eulerian and the Lagrangian approaches (Chopra 1968; Finn and Varoglu 1973; Saini et al. 1978; Chopra and Chakrabarti 1981; Greeves and Dumanoglu 1989; Singhal 1991; Calayir et al. 1996; Bayraktar et al. 1996). Earthquake forces were considered deterministic in these studies. The hydrodynamic effect of reservoir water on dams was modeled with fluid finite elements in both approaches.

There have been few studies concerned with earthquake performance analysis of concrete slabs on CFR dams that include dam-soil-reservoir interaction (Bayraktar and Kartal 2007). When the friction between the concrete slab and rockfill is ignored, the hydrodynamic pressure on concrete slabs of CFR dams is assumed to be inefficient and usually neglected. This paper presents the earthquake responses and performance evaluation of the Torul CFR Dam in empty and full reservoir cases. For this purpose, two-dimensional dam-soil and dam-soil-reservoir finite element models were used. The interface between the concrete slab and rockfill was modeled using interface elements. These elements allow transverse shear deformation between the rockfill and concrete face slab. The ANSYS program was used in finite element analysis. The Lagrangian approach was used to model the impounded water
with two-dimensional fluid finite elements. The Drucker-Prager model was used in nonlinear analysis of the concrete face slab, rockfill and foundation soil.

2 Formulation of dam-soil-reservoir interaction with Lagrangian approach

Dam-soil-reservoir interaction systems are explained in this paper. A fluid system formulation based on the Lagrangian approach is provided by Eq. (1) (Wilson and Khalvati 1983; Calayir 1994). According to the Lagrangian approach, fluid is assumed to be linearly elastic, inviscid and non-rotational. It is also considered compressible in the study. The stress-strain relationship for a two-dimensional fluid can be written as follows:

\[
\begin{bmatrix}
P \\
P_z
\end{bmatrix} = \begin{bmatrix}
C_{11} & 0 \\
0 & C_{22}
\end{bmatrix} \begin{bmatrix}
\varepsilon_v \\
\omega_z
\end{bmatrix}
\]

(1)

where \( P \) is the pressure, which is equal to mean stress; \( C_{11} \) is the bulk modulus; \( \varepsilon_v \) is the volumetric strain of the fluid; \( P_z \) is the rotational stress; \( C_{22} \) is the constraint parameter; and \( \omega_z \) is the angle of rotation about the Cartesian axis \( z \). Rotations and constraint parameters are involved in the stress-strain equation of the fluid, and non-rotationality of the fluid is simulated with the penalty method (Zienkiewicz and Taylor 1989; Bathe 1996).

The equations of motion of the fluid system are obtained using energy principles. The total strain energy of the fluid system can be written as

\[
E_t = \frac{1}{2} U^T f K_f U_f
\]

(2)

where \( U_f \) is the nodal displacement vector and \( K_f \) is the stiffness matrix of the fluid system. The stiffness matrix of the fluid system is obtained with the sum of the stiffness matrices of the fluid elements (\( K_e \)) as follows:

\[
\begin{align*}
K_f &= \sum K_e^f \\
K_e^f &= \int (B_e^T) C_f B_e^v dV^e
\end{align*}
\]

(3)

where \( B_e \) is the strain-displacement matrix of the fluid element; \( C_f \) is the elasticity matrix, which consists of the diagonal terms in Eq. (1), and \( V^e \) is the volume of the fluid elements.

The motion of the fluid system is known as sloshing waves without change of volume in reservoirs and storage tanks. The displacements for the motion are in the vertical direction. Sloshing waves result in an increase in the potential energy of the system as follows:

\[
E_p = \frac{1}{2} U_{sf}^T K_{sf} U_{sf}
\]

(4)

where \( U_{sf} \) is the nodal vertical displacement vector and \( K_{sf} \) is the stiffness matrix of the free surface of the fluid system. The stiffness matrices of the surface fluid elements (\( K_{sf}^e \)) are obtained as follows:
where \( h_f \) is the vector that consists of interpolation functions of the free surface fluid elements, \( \rho_f \) is the mass density of the fluid, \( A^e \) is the area of the free surface fluid element, and \( g \) is the acceleration due to gravity.

The kinetic energy of the system can be written as

\[
E = \frac{1}{2} \dot{U}_f^T M_i \dot{U}_f
\]

where \( \dot{U}_f \) is the nodal velocity vector and \( M_i \) is the mass matrix of the fluid system. The mass matrix of the fluid system is also obtained with the sum of the mass matrices of the fluid elements \( M^e_i \) as follows:

\[
M_i = \sum M^e_i
\]

\[
M^e_i = \rho_f \int H^T H dV^e
\]

where \( H \) is the matrix that consists of interpolation functions of the fluid element. Combining Eqs. (2), (4) and (6) with Lagrange’s equation (Clough and Penzien 1993), the following equation is obtained:

\[
M_i \ddot{U}_f + K^e_i U_f = R_i
\]

where \( K^e_i \) is the system stiffness matrix that includes the free surface stiffness, \( \ddot{U}_f \) is the nodal acceleration vector, and \( R_i \) is the time-varying nodal force vector for the fluid system. A reduced integration scheme is utilized in the formation of the fluid element matrices (Wilson and Khalvati 1983).

The equation of motion of the fluid system, Eq. (8), may be written as those of the structure system. The coupled equations of the fluid-structure system are attained by determining the interface condition. The displacement in the normal direction of the interface is continuous at the system interface, since fluid is assumed to be inviscid. If it is assumed that the normal component of the interface displacement toward the structure \( U^+_n \) is positive, while the normal component of the interface displacement toward the fluid \( U^-_n \) is negative, then \( U^+_n = U^-_n \) (Akkas et al. 1979). Using this condition, the equation of motion of the coupled system, including damping effects of ground motion, is given:

\[
MU + CU + KU = R
\]

in which \( M \) is the mass matrix, \( C \) is the damping matrix, and \( K \) is the stiffness matrix of the fluid-structure interaction system. Also, \( U, \dot{U}, \ddot{U}, \) and \( R \) are the vectors of the displacement, velocity, acceleration, and external load of the coupled system, respectively.

### 3 Structural performance and damage criteria for dams

Linear time-history analysis was used to formulate a systematic and rational methodology
for a qualitative estimate of the level of damage. In linear time-history analysis, deformations, stresses and internal forces are calculated in accordance with elastic stiffness characteristics of various components. When acceleration time-histories are used as the seismic input, both the magnitudes and time-varying characteristics of the seismic response are computed with the linear time-history analysis. A systematic interpretation and evaluation of these results in terms of the demand-capacity ratio \( F \), cumulative inelastic duration, spatial extent of overstressed regions, and possible modes of failure form the basis for estimate of the probable level of damage. The damage to structural performance is measured based on the cracking of the concrete, the opening of construction joints, and the yielding of the reinforcing steel. If the estimated level of damage falls below the acceptance curve for a particular type of structure, the damage is considered low and the linear time-history analysis is sufficient. Otherwise, the damage is considered severe, in which case a nonlinear time-history analysis is required to estimate damage more accurately (Ghanaat 2002).

### 3.1 Performance criteria for linear and nonlinear analysis

The response of the dam to the maximum design earthquake would be unlikely to cause damage if the demand-capacity ratio is less than or equal to 1.0; with this ratio, the response is within what is considered the linear elastic range of behavior. The level of nonlinear response or opening and cracking of joints is considered acceptable if the demand-capacity ratio is less than 2.0, the overstressed region is limited to 15% of the dam surface area, and the cumulative inelastic duration falls below the performance curve (Ghanaat 2002). Cumulative duration has not been defined for the concrete slab of CFR dams up to now; therefore, the performance curve used for concrete gravity dams was considered in this study (USACE 2003).

### 3.2 Demand-capacity ratio

The demand-capacity ratio for gravity dams is defined as the ratio of the calculated principal stress to the tensile strength of concrete. As discussed previously, the demand-capacity ratio is limited to 2.0, thus permitting stresses up to two times the static tensile strength or at the level of the dynamic apparent tensile strength of concrete. Although the tensile strength of concrete is affected by the rate of seismic loading, it is employed to compute the demand-capacity ratio according to the acceptance criteria. Thus a certain level of conservatism is guaranteed in the estimation of damage based on the results of linear elastic analysis.

The cumulative duration beyond a certain demand-capacity ratio is obtained by multiplying the number of times that the stress value exceeds that level by the time-step of the time-history analysis. The cumulative duration in Fig. 1 refers to the total duration of all stress excursions beyond a certain demand-capacity ratio.
4 Mathematical model of Torul CFR Dam

4.1 Torul Dam

The Torul CFR Dam is located on the Harsit River, approximately 14 km northwest of Torul, Gumushane (Fig. 2). The dam construction was completed in 2007 by the General Directorate of State Hydraulic Works. The main goal of the reservoir is power generation. The volume of the dam body is $4.6 \times 10^6$ m$^3$ and the water area of the reservoir at the normal water level is 3.62 km$^2$. The annual total power generation capacity is 322.28 GW. The length of the dam crest is 320 m and the width is 12 m. The maximum height and base width of the dam are 142 m and 420 m, respectively. The maximum water level of the reservoir ($H_{max}$) is 137.5 m. The thickness of the concrete slab is 0.3 m at the crest level and 0.7 m at the foundation level. The lengths of both the reservoir and foundation soil in the upstream direction are considered three times the dam height. The concrete slab has high seepage resistance. The two-dimensional largest cross section and the dimensions of the dam are shown in Fig. 3.
4.2 Material properties

The Torul Dam body consists of a concrete face slab and five rockfill zones: 2A, 3A, 3B, 3C, and 3D, from upstream to downstream. These rockfill zones are arranged from thin granules to thick particles in the upstream-downstream direction. Foundation soil is composed of two layers: limestone (below) and volcanic tufa (upper). The material properties of the dam and soil used in linear and nonlinear analysis are shown in Table 1. The dynamic Young’s modulus values were used in dynamic analysis. The nonlinear analysis procedure was based on the Drucker-Prager model. This model, using the outer cone approximation of the Mohr-Coulomb law (ANSYS 2008), is suitable for granular (frictional) material such as soils, rocks, and concrete. The cohesion and the angle of internal friction of the solid materials are assumed to be 1.225 MPa and 45°, respectively. The concrete slab has a tensile strength of 1.6 MPa and compression strength of 20 MPa. The bulk modulus and density of the reservoir water are 2.07 MPa and 1 000 kg/m³, respectively.

<table>
<thead>
<tr>
<th>Material</th>
<th>Maximum particle size (mm)</th>
<th>Dynamic material property</th>
<th>Dynamic material property</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Young’s modulus (GPa)</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>Concrete face slab</td>
<td>34.20</td>
<td>0.18</td>
<td>2 395.5</td>
</tr>
<tr>
<td>2A (sifted rock or alluvium)</td>
<td>150</td>
<td>14.00</td>
<td>0.26</td>
</tr>
<tr>
<td>3A (selected rock)</td>
<td>300</td>
<td>13.50</td>
<td>0.26</td>
</tr>
<tr>
<td>3B (quarry rock)</td>
<td>600</td>
<td>12.50</td>
<td>0.26</td>
</tr>
<tr>
<td>3C (quarry rock)</td>
<td>800</td>
<td>11.50</td>
<td>0.26</td>
</tr>
<tr>
<td>3D (selected rock)</td>
<td>1 000</td>
<td>10.00</td>
<td>0.26</td>
</tr>
<tr>
<td>Soil (volcanic tufa)</td>
<td>—</td>
<td>8.40</td>
<td>0.40</td>
</tr>
<tr>
<td>Soil (limestone)</td>
<td>—</td>
<td>11.55</td>
<td>0.40</td>
</tr>
</tbody>
</table>

4.3 Finite element model

The two-dimensional dam-soil-reservoir finite element model used in analysis is shown in Fig. 4. In this model, the dam body and foundation soil have 592 and 656 solid finite elements, respectively, impounded water has 495 fluid finite elements, and 16 interface elements are defined between the concrete slab and rockfill. The solid elements used in the analysis have four nodes and 2×2 integration points; the fluid elements have four nodes and 1×1 integration point. Element matrices were computed using the Gauss numerical integration technique (Wilson and Khalvati 1983). A damping ratio of 5% was assumed in finite element analysis according to Rayleigh damping. The foundation soil was assumed to be massless in time-history analysis. The coupling length was set as 1 mm at the reservoir-dam and reservoir-soil interface and 50 numbers of couplings were defined in the dam-reservoir-soil model. The main objective of the coupling process is to maintain equal displacement between the opposite nodes in the normal direction to the interface. The coupling point in the surface normal direction is also shown in the circle in Fig. 4.

ANSYS was chosen for linear and materially nonlinear finite element analysis in this
study. Both solid and contained fluid, as well as interface elements, were included in this finite element program package. Plane42 solid finite elements defined in ANSYS were used for the concrete slab, rockfill and foundation soil. Fluid79 fluid finite elements obtained by modification of the two-dimensional structural solid element (Plane42) were used to model reservoir water. One of the most important steps in modeling fluid elements in ANSYS was selecting a coordinate system in which the vertical coordinates of the fluid elements were negative. In addition, rectangular fluid elements were constructed as much as possible in order to obtain qualified results. Fluid-structure interaction can be modeled using truss elements between the fluid and solid elements or the penalty method. With ANSYS, the performance of this interaction was modeled with the penalty method and also coupled in the surface normal direction.

![Two-dimensional finite element model including impounded water of Torul Dam](image)

Fig. 4 Two-dimensional finite element model including impounded water of Torul Dam

In the above analysis, the height and length of the foundation soil in the downstream direction are taken into account as much as the dam height to sufficiently consider soil effects on dam response.

4.4 Interface element between concrete face slab and rockfill

The earthquake response of the concrete face slab is mostly dependent on its interface with the rockfill. This interface can be modeled as either welded contact or friction contact (Fig. 5). In fact, the concrete slab does not have direct contact with the rockfill. Accordingly, the use of the interface element in finite element analysis can procure more realistic results. This element type allows concrete slabs to slide over the rockfill face (ANSYS 2008). In this study, the transverse shear stiffness of the interface material was considered to be 1.8 MPa/m.

After the construction of the rockfill, shotcrete covers the upstream face of the dam. During the collapse and settlement process, shotcrete cover is deformed and ruptured. Then the surface is smoothed and the concrete slab is covered on this surface, but because of the damage, the shotcrete loses its initial strength. Various studies have been conducted of rock-rock and rock-shotcrete interface stiffness and joints (Wang et al. 2003; Bae et al. 2004).
Interface shear stiffness in these studies was determined for rock-rock and intact shotcrete-rock interfaces. However, these studies do not realistically reflect the related interface; relatively low interface shear stiffness was assumed for concrete-rockfill interfaces in this study.

5 Earthquake response of Torul CFR Dam

The response of the Torul Dam to the horizontal component of the 1992 Erzincan earthquake was evaluated. The duration of the acceleration record of the earthquake is 21.31 s (Fig. 6) and it is available at the PEER Strong Motion Database (PEER 2008). The maximum value of the earthquake acceleration \( a_p \) was 0.515g. The time interval of the acceleration record is 0.005 s.

The maximum displacements in the two opposite directions of the concrete slab were compared by dam height for dam-soil and dam-soil-reservoir systems. In addition, maximum tensile and compression principal stresses were determined for each system by dam height. The changes of the critical horizontal displacement and principal stresses with time were also determined to investigate hydrodynamic effects of the reservoir water on the earthquake response of the dam.

5.1 Displacement

The maximum horizontal displacements \( U_{\text{max}} \) in the two opposite directions of the concrete slab were determined according to linear time-history analysis. Empty and full reservoir cases were taken into consideration. The obtained displacements are shown along the height \( H \) of the concrete slab in Fig. 7. It is clearly seen that the reservoir water leads to the
increase of displacement by the dam height and displacement reaches its maximum value at the crest point of each system.

The time-histories of the maximum horizontal displacements of the concrete slab at the crest level attained from linear analysis are shown in Fig. 8. According to Fig. 8, reservoir water clearly increases the displacement during earthquakes. The deflections of the dam, where the maximum displacements in the two opposite directions occur, are shown in Fig. 9.

5.2 Stress

The maximum principal tensile stress ($\sigma_{T_{\text{max}}}$) and the maximum principal compression stress ($\sigma_{P_{\text{max}}}$) of the concrete slab are shown by dam height in Fig. 10 for the empty and full reservoir cases, respectively. According to Fig. 10, the principal tensile and compression
stresses are increased by reservoir water effects.

![Graphs showing stresses](image)

**Fig. 10** Maximum principal stress on concrete face slab for different systems

The time-histories of maximum principal compression and tensile stresses obtained from linear analysis for both the empty and full reservoir cases are also shown. The effect of reservoir water on the principal stresses is clearly distinguished in Fig. 11. The maximum principal compression and tensile stresses are increased by hydrodynamic pressure for the duration of the earthquake.

![Graphs showing time-histories](image)

**Fig. 11** Time-histories of maximum principal stress on concrete face slab

### 6 Performance analysis of Torul CFR Dam

The performance analysis of the Torul CFR Dam was evaluated in regards to horizontal components of the 1992 Erzincan earthquake. The earthquake performance analysis was carried out for the empty and full reservoir cases to reveal the hydrodynamic effect of reservoir water. First, linear time-history analysis was performed to assess whether the damage to the concrete face slab was acceptable or not, considering the demand-capacity ratio interval from 1.0 to 2.0.

The demand-capacity ratio for each system was considered in order to evaluate the maximum principal tensile stresses on the concrete slab. The maximum principal tensile stresses exceeded the tensile strength of the concrete several times in the empty reservoir case, as seen in Fig. 12(a). However, if hydrodynamic pressure was included in finite element analysis, the excursions of the principal stresses were more than those in the empty reservoir case for the selected demand-capacity ratio interval (Fig. 12(b)).
Fig. 12 Maximum principal tensile stress cycles for linear analysis of different systems

The performance curve is drawn to evaluate whether the linear analysis is sufficient or not. This is required to determine the potential damage level of each system. According to linear analysis, the cumulative duration is completely over the acceptable level for both the empty reservoir case and the full reservoir case. Although the cumulative durations decrease as the demand-capacity ratio increases (Fig. 13), they are still higher than the acceptable level line. The performance curves shown in Fig. 13 indicate that the intense force on the concrete causes it to crack at selected demand-capacity ratio interval. According to the performance estimation of the dam, damage appears to be inevitable. As seen in Fig. 13, the hydrodynamic pressure of the reservoir water significantly reduces the earthquake performance of the dam.

Fig. 13 Earthquake performance evaluation of Torul CFR Dam

Materially nonlinear analysis is necessary for evaluation of the earthquake performance of the dam. According to nonlinear analysis, the maximum principal tensile stresses are small in each case, as seen in Fig. 14. The nonlinear analysis results reveal no cracking in the concrete slab, and therefore no damage will occur to the concrete.

Fig. 14 Maximum principal tensile stress cycles for nonlinear analysis of different systems
7 Conclusions

There have been few studies of earthquake performance of the concrete slab of a CFR dam that involve friction contact between the concrete slab and rockfill in a dam-soil-reservoir interactive system subjected to strong ground motion. This paper presents earthquake performance analysis of the Torul CFR Dam in empty and full reservoir cases. The earthquake performance of the concrete is evaluated with consideration of interface elements between the concrete slab and rockfill to obtain numerical results more realistically. The Lagrangian approach was used to examine the effect of reservoir water in the finite element analysis.

The earthquake response of the dam is clearly affected by the hydrodynamic pressure of reservoir water. Hydrodynamic pressure increases the maximum displacements in two opposite directions as well as the maximum principal stresses during an earthquake.

According to linear time-history analysis of the dam, the earthquake caused momentous damage to the concrete slab. Hydrodynamic pressure played an important role in reducing the earthquake performance of the dam. Therefore, materially nonlinear time-history analysis was performed, and the result showed that no damage occurred in either reservoir case.

Some suggestions are as follows:

1. A more realistic earthquake response and performance assessment of CFR dams can be carried out using interface elements capable of accounting for the transverse shear deformation.

2. The hydrodynamic pressure of reservoir water should be taken into account in earthquake performance analysis of modern CFR dams.

3. Nonlinear earthquake analysis should be performed to evaluate earthquake performance of CFR dams. The results will be more reliable.

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References


