Effect of Foundation-Reservoir Interaction on Seismic Response of Concrete Gravity Dam

R. Ayothisram4, Damodar Maity2 and Garnish Kashung3

Key words
Seismic response, gravity dam, dam-foundation interaction, hydrodynamic effect, numerical analysis, case study

Abstract: Effect of foundation-reservoir interaction on seismic response of concrete gravity dam is investigated considering Bichom Dam located in Arunachal Pradesh, India. Seismic analysis is carried out for two earthquake excitations (DBE and MCE) assuming linear behaviour. Time period, crest displacement, base reactions and stress distributions of dam are discussed herein. The dam with rigid foundation subjected DBE excitation is safe except some minor cracks at the heel, but the dam with flexible foundation suffers moderate damage when the reservoir is empty and full. The dam subjected MCE excitation experiences serious cracks at heel and toe of the dam.

Introduction

The safety of concrete gravity dams to seismic excitation has been the subject of research as the risk posed by the increasing population in the downstream location of dams is growing and the advancement of earthquake engineering knowledge reveals the inadequacy of the seismic design concepts used at that time. It has been observed that damage of concrete dams could occur for earthquake intensity that are less than the maximum value that could be expected at a site. Such damage or failure of dam structure would lead to disastrous consequences for both life of human and the environment. Hence seismic analysis of dams considering the complex interactions that would occur under seismic loading has been receiving considerable attention. The seismic response of gravity dam is influenced by various factors; characteristics of dam, dam-site, foundation and earthquake excitation and hydrodynamic effect. Gogoi and Maity (2005) presented the state-of-the-art related to stability analysis of concrete dams. Earlier investigations (Westergaard, 1933; Werner and Sundquist, 1949; Zangar, 1950; Zangar and Haefeli, 1952; Kotsubo, 1959; Bustamante and Flores, 1966; Hatano, 1965; Chopra, 1967; Flores et al., 1969; Clough et al., 1987; Zee and Zee, 2006) accounted for the effect of hydrodynamic water pressure in addition to hydrostatic pressure on the response of rigid dams under earthquakes and estimated the influence of inclination of upstream face of the dam and compressibility of reservoir water. Chopra and his co-workers developed methods to examine the importance of considering dam-reservoir interaction (Chopra, 1967; Chopra, 1968; Chopra, 1970; Rea et al., 1975; Hall and Chopra, 1982a; Hall and Chopra, 1982b) and dam-foundation-reservoir interaction (Chopra et al., 1980; Chopra and Chakrabarti, 1981) in the seismic response of concrete gravity dam. With the development of finite element method and advances in dynamic analyses, Back et al. (1969) modelled dam as an assemblage of three-dimensional finite elements with a mesh of incompressible liquid elements and the dynamic response was calculated by the mode-superposition method. Finite element and finite difference methods have been successfully in the last few decades by various authors (Parrinello and Borino, 2007; Pekau and Zhu, 2006; Zhu and Pekau, 2007; Fitma and Leger, 2006; Leger and Javanmardi, 2007; Gogoi and Maity, 2007; Bougacha and Tassoulas, 2006; Maeso et al., 2004; Maity and Bhattacharya, 2003) accounting for the effects of dam-water interaction, dam-foundation interaction and effect of sediments on the seismic response of gravity dams. Hall (1986) studied the earthquake response of Pine Flat Dam subjecting to a large set of time history of acceleration considering the various effects such as presence of water and its compressibility and the vertical component of ground motion. It was found that the presence of water significantly increases the earthquake response of concrete gravity dam, water compressibility does not, by itself; greatly increase the maximum dam response. El-Aidi and Hall (1989) examined the dam-water-foundation interaction with emphasis on the non-linear behaviour associated with concrete cracking and water cavitation. It was shown that water cavitation has little effect on the dam response. However concrete cracking plays a significant role. Similar investigations on the seismic response of cracked dams and effect of crack penetration on seismic response have been reported by various authors (Pekau and Zhu, 2006; Zhu and Pekau, 2007; Fitma and Leger, 2006). Chavez and Fenves (1995) studied the stability of a concrete dam against sliding along the interface between the dam base and the foundation rock and concluded that, it is necessary to include the effects of dam-foundation interaction to

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obtain realistic estimates of the base sliding displacement for a dam and water compressibility also affects the base sliding displacement. The sliding displacement is sensitive to the value of the coefficient of friction for the interface zone, especially moderate to tall dams. It is well known that sediments are deposited at the base of the reservoir on upstream side which significantly alters the response of the dam under seismic loading. The effect of sediments on the seismic response of gravity dams were investigated by various authors (Gogoi and Maity, 2007; Bougacha and Tassoulas, 2006; Maeso et al., 2004; Lotfi et al., 1987; Medina et al., 1990; Cheng, 1986; Bougacha and Tassoulas, 1991a, 1991b & 1991c; Dominguez, Gallego and Japo n, 1997; Maeso et al., 2002). The studies on the effect of sediments in earlier stage (Lotfi et al., 1987; Cheng, 1986; Lin and Tassoulas, 1987) reported that the effect of sediment on seismic response of gravity and arch dams is very small and negligible. However it is found by other researchers at later stage that the partially saturated sediment can cause substantial changes in the seismic response of gravity and arch dams by affecting the dynamic characteristics of the system. It is also reported that the effect of sediments is function of frequency, particularly fundamental frequency and frequency content of seismic excitation. Based on the critical assessment of literature, it is found that the dynamic response of dams to earthquake excitation depends on several factors including intensity and characteristics of the design earthquakes, interaction of the dam with the foundation rock, reservoir water, and sediments thus a complete analysis of seismic response of concrete gravity dam must include the effects of dam-foundation rock-fluid-sediment interaction. It is also reported in the literature that the significance of effects of these interactions depends on the dam-site and dam conditions, which may play a predominant role. Depending on rigidity of foundation rock, sectional details of dam, rate of sedimentation and type and conditions of sedimentation materials, the effect of dam-reservoir-foundation-sediment interaction may be significant or may not be also. Hence, it is to be noted that the seismic response of dam is case-specific and thus requiring analysis of each dam considering the all possible interactions. This paper presents the findings of an investigation on effect of coupled foundation-reservoir interaction on the seismic response of concrete gravity dams, through a case study: Bichom dam across Bichom River in Arunachal Pradesh, North-Eastern India.

Case Study Details

The Kameng Hydroelectric Project of 600 MW located in Kameng District of Arunachal Pradesh, India envisages the construction of two concrete gravity dams viz. Bichom and Tenga. The project site is in North-Eastern India which is a seismically active area (Zone-V) as per IS 1893 (2002) and hence seismic analysis and design of these dams are mandatory. Seismic analysis of Bichom dam is considered in this case study. The catchment area of the Bichom dam is 2277 sq. km and the design flood discharge is 10476.40 cumecs. The Bichom dam has full reservoir level (FRL) at EL 770 m and maximum water level (MWL) at 772.5m. The dam is a concrete gravity type with maximum height of 96.5 m above the deepest foundation level. The total length of the dam is 200 m and consists of 7 non-overflow monoliths and spillway (overflow) monoliths each. The schematic layout of the Kameng Hydroelectric Project is shown in Figure 1.

Seismic Analysis

The seismic response of Bichom Dam in time domain is carried out for the estimated time-history of acceleration of Design Basis Earthquake (DBE) and Maximum Credible Earthquake (MCE) considering the site effects. Linear elastic behaviour is assumed. Seismic analysis is carried out using a three-dimensional finite element model by which potential modes of failure can be identified and stability of the piers can be assessed. For this purpose, finite element software, SAP 2000 (2004) is used. The analyses are carried out for an empty and full reservoir condition. To investigate the influence of Foundational flexibility effects, the dynamic response of the dam is performed assuming that the dam is founded on rigid foundation and flexible foundation. The material properties of the dam and foundation rock are selected from the geotechnical investigation report.

Earthquake Excitation

The earthquake excitations were estimated considering the geology of the region, local geology around the site, earthquake occurrence in the region and the seismotectonic set-up of the area. Among the different earthquake excitations, Maximum credible earthquake (MCE) and Design basis earthquake (DBE) are considered for the case study. The parameters for MCE are generated based on deterministic hazard analysis considering 11 potential sources. The peak ground acceleration was estimated using the empirical attenuation relationship given by Abrahamson and Liehiser (1989),

\[
\log(a) = -0.62 + 0.177M - 0.982\log(r + e^{0.284M}) + 0.132F - 0.0008E \quad (1)
\]

where \(a\) is peak horizontal acceleration, \(r\) is the distance in km to the closest approach of the zone of energy release, \(M\) is the magnitude, \(F\) and \(E\) are dummy variables depending on types of fault and earthquake events. Site-specific ground motions parameters are arrived at based on seismic hazard analysis (IIT Roorkee, 2001) and the normalized time history of
**Fig. 1** Layout of Kameng hydroelectric project

**Fig. 2** Site-specific normalized time history of acceleration for Kameng project site
acceleration for different earthquakes is shown in Figure 2. The ordinate of the Figure 2 is multiplied with 0.37g to get MCE excitation time history for Kameng site. Using appropriate reduction factor of 2 with respect to MCE, the design basis earthquake is evaluated. The ordinate of the Figure 2 of time history of acceleration is multiplied with 0.155g to obtain DBE excitation for the Kameng site.

Modeling of Dam-Foundation-Reservoir System

The Bichom dam consists of seven non-overflow monoliths and seven overflow monoliths with similar geometry that are expected to respond similarly to static and dynamic loads in general. But each monolith tends to resist loads independently with little support from the neighboring monoliths on either side, which is ignored in this study. The modeling of overflow monolith and non-overflow monolith has been done separately and seismic analyses have been carried out considering foundation-reservoir interaction in each section in SAP 2000 (2004).

Size of Foundation Model

A complete foundation model requires very large size mesh where boundary effects on the stresses in the dam become negligible. The foundation model is chosen such that there is not much variation in the response of the dam with the change of the foundation size. Based on the preliminary analysis performed with different sizes of foundation, the foundation size of $5b \times 2b$ is found to give acceptable response, where $b$ is the base width of the dam. Therefore, by providing symmetric and anti-symmetric boundary conditions at the sides of a single monolith, its deflection and stresses can be computed independently. The dam section is modeled by an assembly of 8-noded solid elements. In addition to the conventional boundary conditions under static loads and absorbing boundary conditions under seismic loads are additionally required. These boundary conditions are modelled as per the procedure suggested by Gogoi and Maity (2007). The typical finite element mesh of overflow monolith and non-overflow monolith with rigid foundation is shown in Figure 3.

Material Properties

Concrete

The concrete mass in the dam is assumed to be homogeneous, isotropic, linear elastic. In both monoliths i.e. overflow and non-overflow monoliths, the grade of concrete is M15 except at the edges of around 2 m which is M20 grade. The unit weight of concrete ($\gamma_c$) is taken as 24 kN/m$^3$, Poisson’s ratio of concrete ($\nu$) of 0.20 and seismic modulus of elasticity of concrete ($E_c$) as 25670 MN/m$^2$. Energy dissipation in the dam is represented by a viscous damping ratio of 5% in all natural vibration modes of the dam.

Foundation-Rock

The foundation rock is idealized by a homogeneous, isotropic and linear elastic solid. From the geotechnical investigation report, three kinds of rock were found at the site of the dam viz. phyllite rock, schist rock and gneiss rock. The rock properties were obtained from the geotechnical report and the mean value of these three rocks has been used in the analysis assuming the rock as homogeneous one. The mean rock properties are: Young’s Modulus of Elasticity, $E_r = 36410$ MN/m$^2$, and Poisson’s ratio of foundation rock ($\nu$) is 0.33. The unit weight of foundation rock ($\gamma_r$) is taken as 26 kN/m$^3$. A constant hysteretic damping factor of 0.10 is assumed.

Hydrodynamic Effect

The water in the reservoir impounded by the dam and its hydrodynamic effect is modeled by Chopra’s simplified method (Hall and Chopra, 1982). It is represented as an assemblage of added mass based on this method. This added mass representation is based on the fundamental mode and it accounts for the compressibility of water and the interaction of water with the elastic structure and foundation. Even in recent times, the added mass approach has been used successfully for modelling fluid-structure interaction.
(Livaoğlu and Doğangün, 2006); hence the same method is used in this study also. The maximum water level in the overflow section and non-overflow section are 67.25 m and 67.81 m respectively. The unit weight of water is taken as 10 kN/m$^2$ and the velocity of pressure waves, C as 1438 m/s. To account the effect of reservoir bottom absorption, the wave reflection coefficient ($\alpha$) of 0.74 was used. The initial and final added mass of overflow section and non-overflow section were determined based on the geometry and material properties of the dam. These values were used in the analysis to account for the hydrodynamic effect.

**Validation**

A benchmark problem (Yazdchi et al., 1999) has been solved based on the proposed methodology using the finite element software, SAP 2000 (2004) and results have been compared by one of the coauthors of this paper in their earlier work (Reddy et al., 2008). Yazdchi et al. (1999) solved this problem by the coupled FEM-BEM method whereas present method is dealt with FEM-FEM for both dam and foundation. Therefore, a little variation of the results is observed.

**Analysis**

Analyses have been carried out for rigid and flexible foundations and by considering the empty reservoir and full reservoir conditions. The time-history modal superposition method is used. Modal properties were computed using Ritz Vectors for more efficiency. The seismic response such as time period, crest displacements, stresses and base reactions at critical section of the dam are computed for different cases and are discussed below.

**Results and Discussion**

The response of overflow monolith and non-overflow monolith such as mode shape and time period, crest displacement, base reactions and stresses at the heel and toe of the dam due to DBE and MCE excitations obtained from the analysis is presented and discussed herein. The typical mode shape of overflow monolith and non-overflow monolith at fundamental natural frequency for dam with empty reservoir condition is shown in Figures 4 and 5 respectively. It is observed from the figure that the crest of the dam displaces in a same direction at fundamental mode for dam and similar response at fundamental mode was observed for dam with flexible foundation and full reservoir condition.

**Time Period**

The time period of overflow monolith and non-overflow monolith up to twelve modes and for the different cases: (a) Empty reservoir with Rigid foundation (ER), (b) Empty reservoir with Flexible foundation Full reservoir with Rigid foundation (EF), (c) Full reservoir with Rigid foundation (FR) and (d) Full reservoir with Flexible foundation (FF) is summarized in Table 1. It is found from the Table 1 that the time period of overflow monolith with mode number is similar for both rigid and flexible foundation. However, the time period of dam with flexible foundation is larger than that of the dam with rigid foundation at all modes. This clearly shows that the time period is significantly influenced by the foundation flexibility. The increase of time period for flexible foundation is due to less stiffness of foundation. It is also found from the figure that the reservoir condition (empty/full) has only marginal effect on the time period of dam-foundation system. The foundation flexibility increases the fundamental time period by 15%, whereas the coupled foundation-reservoir interaction increases the fundamental time period by 20%. This shows that the combined effect of foundation flexibility and full reservoir condition (hydrodynamic effect) is significant on the fundamental time period of overflow monolith of concrete gravity dam. Similar observation is noticed for non-overflow monolith. However, it is observed from Table 1 that the coupled foundation-reservoir interaction increases the fundamental time period by 50% for non-overflow section.

As the cross section of overflow monolith and non-overflow monolith are entirely different and of different height, the fundamental time period is also varying. In both case of foundation condition, the time period of non-overflow monolith is lesser than the overflow monolith. For instance, the fundamental time period of overflow section for an empty reservoir of rigid foundation is 0.2453 sec, while for non-overflow section is 0.1562 s. This indicates that the non-overflow section is comparatively stiffer than the overflow section. It is also found from the results that the dam-water-foundation rock interaction lengthens the fundamental resonant period of the non-overflow monolith.

**Crest Displacement**

The typical time history of crest displacement obtained from the analysis for a dam (overflow monolith and non-overflow monolith) with flexible foundation and empty reservoir condition is shown in Figures 6 and 7 respectively. It is inferred from the figure that the time history of displacement response of overflow monolith to DBE and MCE looks exactly similar, but the amplitude of crest displacement under MCE is larger than the displacement under DBE. The reason for this is obvious that the intensity of shaking in MCE is larger than DBE. It is also noted from the Figure 6 that the peak displacement is occurring within 10 to 20 sec of the excitation. The maximum crest displacement measured from the time histories of displacement for different
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**Fig. 4** First mode shape of overflow section with rigid foundation and empty reservoir condition

**Fig. 5** First mode shape of non-overflow section with rigid foundation and empty reservoir condition.

**Fig. 6** Typical time history of crest displacement for overflow monolith with flexible foundation and empty reservoir.

**Fig. 7** Typical time history of crest displacement for non-overflow monolith with flexible foundation and empty reservoir.
### Table 1 Summary of modal analysis

<table>
<thead>
<tr>
<th>Mode No</th>
<th>Overflow Monolith</th>
<th>Non-Overflow Monolith</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ER</td>
<td>EF</td>
</tr>
<tr>
<td>1</td>
<td>0.245</td>
<td>0.281</td>
</tr>
<tr>
<td>2</td>
<td>0.090</td>
<td>0.158</td>
</tr>
<tr>
<td>3</td>
<td>0.080</td>
<td>0.150</td>
</tr>
<tr>
<td>4</td>
<td>0.050</td>
<td>0.122</td>
</tr>
<tr>
<td>5</td>
<td>0.045</td>
<td>0.100</td>
</tr>
<tr>
<td>6</td>
<td>0.038</td>
<td>0.089</td>
</tr>
<tr>
<td>7</td>
<td>0.035</td>
<td>0.082</td>
</tr>
<tr>
<td>8</td>
<td>0.030</td>
<td>0.068</td>
</tr>
<tr>
<td>9</td>
<td>0.025</td>
<td>0.065</td>
</tr>
<tr>
<td>10</td>
<td>0.022</td>
<td>0.045</td>
</tr>
<tr>
<td>11</td>
<td>0.018</td>
<td>0.038</td>
</tr>
<tr>
<td>12</td>
<td>0.015</td>
<td>0.024</td>
</tr>
</tbody>
</table>

### Table 2 Summary of response of overflow monolith to DBE

<table>
<thead>
<tr>
<th>Response</th>
<th>Rigid Foundation</th>
<th>Flexible Foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Empty Reservoir</td>
<td>Full Reservoir</td>
</tr>
<tr>
<td>Crest Displacement (mm)</td>
<td>7.73</td>
<td>9.38</td>
</tr>
<tr>
<td>Base Shear (kN)</td>
<td>15,146</td>
<td>18,056</td>
</tr>
<tr>
<td>Base Moment (kN-m)</td>
<td>644,396</td>
<td>732,208</td>
</tr>
<tr>
<td>Maximum Principal Stress</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(kN/m²)</td>
<td>Heel</td>
<td>1271</td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>824</td>
</tr>
<tr>
<td>Minimum Principal Stress</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(kN/m²)</td>
<td>Heel</td>
<td>1354</td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>774</td>
</tr>
</tbody>
</table>

### Table 3 Summary of response of overflow monolith to MCE

<table>
<thead>
<tr>
<th>Response</th>
<th>Rigid Foundation</th>
<th>Flexible Foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Empty Reservoir</td>
<td>Full Reservoir</td>
</tr>
<tr>
<td>Crest Displacement (mm)</td>
<td>15.47</td>
<td>18.76</td>
</tr>
<tr>
<td>Base Shear (kN)</td>
<td>30,290</td>
<td>36,110</td>
</tr>
<tr>
<td>Base Moment (kN-m)</td>
<td>1288,750</td>
<td>1464,368</td>
</tr>
<tr>
<td>Maximum Principal Stress</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(kN/m²)</td>
<td>Heel</td>
<td>2541</td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>1649</td>
</tr>
<tr>
<td>Minimum Principal Stress</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(kN/m²)</td>
<td>Heel</td>
<td>2708</td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>1549</td>
</tr>
</tbody>
</table>
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Table 4 Summary of response of non-overflow monolith to DBE

<table>
<thead>
<tr>
<th>Response</th>
<th>Rigid Foundation</th>
<th>Flexible Foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Empty Reservoir</td>
<td>Full Reservoir</td>
</tr>
<tr>
<td>Crown Displacement (mm)</td>
<td>4.73</td>
<td>6.89</td>
</tr>
<tr>
<td>Base Shear (kN)</td>
<td>10,891</td>
<td>17,026</td>
</tr>
<tr>
<td>Base Moment (kN-m)</td>
<td>299,114</td>
<td>459,504</td>
</tr>
<tr>
<td>Maximum Principal Stress (kN/m²)</td>
<td>Heel</td>
<td>954</td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>75</td>
</tr>
<tr>
<td>Minimum Principal Stress (kN/m²)</td>
<td>Heel</td>
<td>876</td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>120</td>
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</tbody>
</table>

Table 5 Summary of response of non-overflow monolith to MCE

<table>
<thead>
<tr>
<th>Response</th>
<th>Rigid Foundation</th>
<th>Flexible Foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Empty Reservoir</td>
<td>Full Reservoir</td>
</tr>
<tr>
<td>Crown Displacement (mm)</td>
<td>9.47</td>
<td>13.79</td>
</tr>
<tr>
<td>Base Shear (kN)</td>
<td>21,782</td>
<td>34,052</td>
</tr>
<tr>
<td>Base Moment (kN-m)</td>
<td>598,229</td>
<td>918,978</td>
</tr>
<tr>
<td>Maximum Principal Stress (kN/m²)</td>
<td>Heel</td>
<td>1907</td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>151</td>
</tr>
<tr>
<td>Minimum Principal Stress (kN/m²)</td>
<td>Heel</td>
<td>1752</td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>240</td>
</tr>
</tbody>
</table>

cases are tabulated in Tables 2 to 5. It can be seen from Table 2 and 3 that for overflow monolith with E.R under DBE, the crest displacement is 7.73 mm and 15.47 mm for MCE, i.e. two times that of DBE. This observation is consistent for other cases also (E.F, F.R and F.F). It is also inferred from Table 2 & 3 that the crest displacement is substantially increasing when the foundation is flexible for same earthquake excitation; however the increase in crest displacement with full reservoir condition is only marginal. It is also seen from Tables 2–5, it can be seen that the maximum horizontal crest displacement experienced in non-overflow monolith is lesser than that of overflow monolith for rigid foundation, due to high stiffness of non-overflow section.

But, if foundational flexibility is considered then the crest displacement of both monoliths are very near to each other, which indicate that even stiffer sections of the dam experiences larger crest displacement when the foundation is flexible, i.e. when dam-foundation interaction is considered. It is seen from Tables 2—5 that the dam response for MCE motion is double of DBE motion for moth monoliths with different foundation flexibility and reservoir loadings.

Base Reactions

The base shear and base moment for DBE and MCE are also measured from the analysis and are presented in Tables 2 to 5 for both monoliths and various cases. For all cases of reservoir loading and foundation flexibility, the magnitude of response of the base reactions for MCE is more than two times of the DBE. However, the overturning and stability criteria has little meaning in the context of the oscillatory response during the earthquakes as other criteria such as exceeding permissible stress (resulting to cracks) will respond earlier. Hence, the analysis of stress distribution in the dam section is important rather than the external stability analysis for seismic loads, which is discussed in next section.
Stress Distribution

The concentration of stresses at the heel and toe of the dam are measured from the analysis. As there is a pier structure in the overflow monolith for the purpose of erecting steel gates, the concentration of stress at their interface will also be important. The major principal stresses shows the concentration of compressive stress while the minor principal stress indicates the tensile stress experienced at the section. Two grades of concrete M15 and M20 are mainly used while other higher grades are used at the drainage or inspection gallery. As described in material properties section, M20 are used at the outer parts/edges of the dam while M15 are used in the interior parts of the dam. The permissible compressive strength of the concrete at heel and toe are estimated from M20 i.e. 20 N/mm² or 20000 kN/m². The tensile strength can be calculated as given by the Indian standard, IS 456 (2000), \( f_{ct} = 0.7 \sqrt{f_{ck}} \text{ N/mm}^2 \); where \( f_{ct} \) is the characteristic strength of concrete in N/mm².

However, according to the criteria as specified in the report by University of Roorkee (2001), for concrete dams, the maximum tension under DBE may be allowed to exceed upto 12.5% of the ultimate compressive stress. While for MCE the maximum tension should be allowed to exceed 50% more than those specified for DBE but not exceeding 2000 kN/m². Based on these criteria, the permissible tensile strength for DBE and MCE is estimated as 2500 kN/m² and 2000 kN/m² respectively which are used in this study.

The typical time histories of maximum principal stress (i.e. compressive stress) measured at heel off the overflow monolith and non-overflow monolith with flexible foundation are shown in Figures 8 and 9 respectively. The maximum and minimum principal stresses measured from the analysis for different cases are summarized in Tables 2 to 5 for both monoliths under DBE and MCE excitations.

![Fig. 8 Time history of stresses at heel of overflow monolith with EF condition under DBE](image)

![Fig. 9 Time history of stresses at heel of non-overflow monolith with EF condition under DBE](image)

In case of overflow monolith, the permissible compressive strength of concrete is very large there would be no failure of dam due to compressive force. The heel and toe of the dam are safe under DBE against compressive force. The tensile stress of concrete is exceeding the permissible limit at the heel for both loading case of empty and full reservoir on dam with flexible foundation. The implication of this exceeding tensile stress above the permissible limit can be examined with its duration. If the duration of exceeding the permissible stress is longer then it will result in major cracking of concrete. But, it is observed from time histories of stresses that the time duration of exceeding the permissible stress is very short. The positive ordinate represents the compressive stresses while the negative ordinate indicates the tensile stresses. In both the loading cases, the time duration of crossing the permissible tensile stress at the heel is short. As such they are not capable of generating sufficient energy to extend the cracks through the entire base section. The tensile stress at the toe under full reservoir loading is also experiencing the overstressing for a short duration. The heel and toe of the dam (overflow monolith) are expected to suffer minor cracks under DBE motion. The distribution of minimum principal stress for rigid foundation also follows similar pattern. Although the hydrodynamic effect increases relatively the magnitude of stresses in the full reservoir loading, the stress concentration at the heel and toe of dam significantly increases when foundation flexibility along with hydrodynamic effects are considered. The excitation characteristics of both DBE and MCE are same except that MCE is higher in magnitude. Thus, all the peak responses occurring at a particular time in DBE also occurs in MCE. It is found from the Table 3 that for MCE excitation, the stress concentration is much larger and exceeds the tensile strength of the concrete for all loading cases both on rigid and flexible foundation. For MCE excitation, the maximum tensile stress experienced at the heel for empty reservoir of rigid foundation is 2708 kN/m² while for the flexible foundation is 6524 kN/m². This indicates that the response obtained for rigid foundation is underestimated as compared to flexible foundation. This difference is also seen when
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hydrodynamic effects are considered. In case of rigid foundation, the time duration of tensile stress exceeding the permissible stress is quiet short that it may result only in minor cracks and it may not be possible to propagate it further. However, the time duration of exceeding the permissible tensile stress is longer in the flexible foundation. This would have an impact on the potentiality of generating sufficient energy to extend the cracks through the entire base section. The maximum tensile stress observed at the heel for empty reservoir is 6524 kN/m². This critical stress concentration continues for around 24 sec at the range of 3000 kN/m². For full reservoir the maximum tensile stress computed is 7263 kN/m² and continues to experience the exceeded tensile stress of around 3500 kN/m² till 24 sec of the excitation. Thus, the heel will suffer major cracks. This trend is also observed for the toe in flexible foundation case although the stress concentration is lesser as that in heel. But the tensile stress experienced is significant enough to suffer major cracks and is capable of extending it. The tensile stress at toe for empty reservoir is 3762 kN/m² and 4611 kN/m² for full reservoir with flexible foundation. This indicates that both heel and toe of the overflow monolith experiences major cracks, which has capability to extend further. But, this behaviour can not be predicted in this study, since it has assumed linear behaviour. To investigate the more realistic behavior and performance of the monolith during the critical MCE excitation it would be necessary to consider nonlinear analysis.

In case of non-overflow monolith, it is observed from Table 4 that the concentration of compressive stress due to DBE excitation is less than the permissible stress. Hence, the non-overflow monolith is also safe from the compressive failure of concrete cracking. The monolith is safe as the tensile stress concentration is below the permissible limit at the heel and toe except in the flexible foundation case. The tensile stress experienced at the heel for empty reservoir and full reservoir are 3358 kN/m² and 4590 kN/m² respectively for DBE excitation. Thus it is above the permissible tensile strength of the concrete and it will result to initiation of concrete cracking. The time duration of exceeded tensile stress at the heel in empty reservoir is not long enough. The stress is below 2500 kN/m² (permissible stress) all along the significant period and reaches the highest stress and reaching 3358 kN/m² at 15.12 sec after the excitation and reduces again below the limit. So it would not result in major cracking. But when the hydrodynamic effects are considered then the intensity of tensile stress as well as duration of this intensity proves to be critical. Even before reaching the maximum tensile stress at 12.1 sec, the average exceeded stress is above 2500 kN/m² lasting for about 25 sec during the excitation. Therefore, this would result in extending the cracks along the section. It is found from Tables 4 & 5 that for DBE excitation, only the heel experiences the exceeded tensile stress in flexible foundation only, whereas, for MCE excitation, the tensile stress is exceeding beyond the permissible limits at the heel for both rigid and flexible foundations. It could be inferred from Table 5 that even the stress concentration at toe is also alarming as it is just below the permissible stress. It is seen that the stress is much above this limit at the average of 2300 kN/m² for around 15 sec. Thus, this will result in major cracking and is capable of propagating it along the section. It is obvious that the duration of period for this exceeded tensile stress limit for empty reservoir will result in major cracking as it is experiencing above 3000 kN/m² for more than 20 sec. This will result in further extension of cracks along the base. Most apparently the cracks at the heel due to tensile stress for full reservoir will be the worst as it is continually propagating above 4000 kN/m² for around 24 sec. The cracks in this condition will be the worst and is much severe than the overflow section. In order to evaluate the real behavior and performance of the concrete under MCE excitation, a nonlinear analysis will be essential as the non-linear property of concrete will be included to show the performance of cracked concrete dam.

Conclusions

The seismic behavior and performance of Bichom Dam when subjected to Design Basis Earthquake and Maximum Credible Earthquake excitations have been analyzed using a three-dimensional finite element model. The conclusions arrived from the findings of present study are presented below:

- The influence of foundational flexibility and hydrodynamic effect on the seismic response of concrete gravity dam is significant, which need to be considered rigorously in seismic analysis of dam structures.

- For Bichom Dam, the fundamental time period of overflow monolith is larger than that of non-overflow monolith, which shows that non-overflow monolith is stiffer that overflow monolith. Foundation-reservoir interaction lengthens the fundamental time period significantly.

- The crest displacements of the two sections are found to be different for different loading and foundation condition. However, when the complete interactions are considered, the maximum crest displacement for both overflow and non-overflow monoliths is found to be same. The base reactions of the dam (base shear and base moment) are amplified when the foundation is flexible.

- When the dam foundation is assumed to be rigid, the dam is completely safe for DBE excitation. However, if dam-foundation interaction is
considered, then the heel of the dam may experience minor cracks for an empty reservoir, but the heel of non-overflow section may undergo major cracks when the reservoir is full.

- Dam subjected to MCE excitation may experience major cracks and there is a potential for extension of cracks along the base of the section for both overflow and non-overflow monoliths. The intensity of these cracks seems to be significantly serious as the tensile stress experienced is much above the permissible tensile strength for some significant duration. Since linear analysis is performed in the present study, a valid estimate of the damage and crack propagation in the dam could not be made. Thus, a rigorous estimate of the seismic safety of Bichon dam could be obtained by performing a nonlinear seismic analysis, which will be future study.

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References


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R. Ayothiraman, Damodar Maity and Garnish Kashung


IIT Roorkee (2001): ‘A report on site specific earthquake parameters for Kameng and Damwe hydroelectric project sites, Arunachal Pradesh’, Department of Earthquake Engineering, University of Roorkee, India.


