Effect of Foundation-Reservoir Interaction on Seismic Behaviour of Gravity Dams

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6th International Conference on
Case Histories in Geotechnical Engineering, Arlington, VA, August 11-16, 2008

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ABSTRACT

This paper discusses the combined effect of foundation-reservoir interaction on the seismic response of concrete gravity dam by considering a case study: Bichom Concrete Gravity Dam located in Arunachal Pradesh, India. The dam comprises of overflow and non-overflow monoliths and seismic analysis was carried out for both monoliths separately for Design Basis Earthquake excitation (DBE) assuming linear behaviour. The significance of foundation flexibility on the seismic response of dam was investigated by comparing the response of dam with rigid and flexible foundations. The hydrodynamic effect of impounded water is modeled as an added mass by Chopra's Method. Effect of foundation-reservoir interaction on the response of both monoliths, such as time period, crest displacement, base reactions and stress distributions are discussed in this paper. It is predicted from the analysis that the dam with rigid foundation is relatively safe except some minor cracks at the heel of non-overflow monolith, but the dam with flexible foundation suffers moderate damage when the reservoir is empty and full.

INTRODUCTION

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 for more than three decades. 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 (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, 1970; Rea et al. 1975; Hall and Chopra, 1982) and dam-foundation-reservoir interaction (Chopra et al. 1980; Chopra and Chakrabarti, 1981) in the seismic response of concrete gravity dam. Numerical methods have been successfully adopted in the last few decades by various authors (Hall, 1986; Maity and Bhattacharyya, 2003; Maeso et al. 2004; Bougacha and Tassoulas, 2006; Ftima and Leger, 2006; Pekau and Zhu, 2006; Gogoi and Maity, 2007; Leger and Javanmardi, 2007; Parrinello and Borino, 2007; Zhu and Pekau, 2007) accounting for the effects of dam-water interaction, dam-foundation interaction and effect of sediments on the seismic response of gravity dams. This paper discusses combined effect of foundation-reservoir interaction on the seismic response of concrete gravity dam by considering a case study: Bichom Concrete Gravity Dam located in Arunachal Pradesh, North-Eastern India.

DETAILS OF CASE STUDY

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 zone (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 Fig. 1.

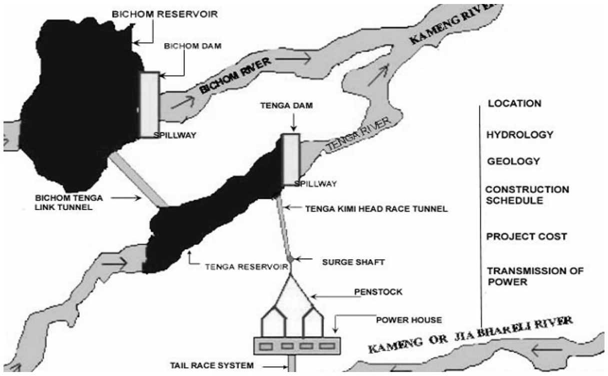


Fig. 1. Layout of Kameng hydroelectric project.

EARTHQUAKE EXCITATION

The earthquake excitation was 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, Design basis earthquake (DBE) was considered for the case study.

The parameters for estimation of earthquake excitation were generated based on deterministic hazard analysis considering 11 potential fault sources. The peak ground acceleration was estimated using the empirical attenuation relationship given by Abrahamson and Litehiser (1989),

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

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 motion parameters were arrived at based on seismic hazard analysis (University of Roorkee, 2001) and the normalized time history of acceleration is shown in Fig. 2. The ordinate of the Fig. 2 is multiplied with 0.155g to get DBE excitation time history for Kameng site.

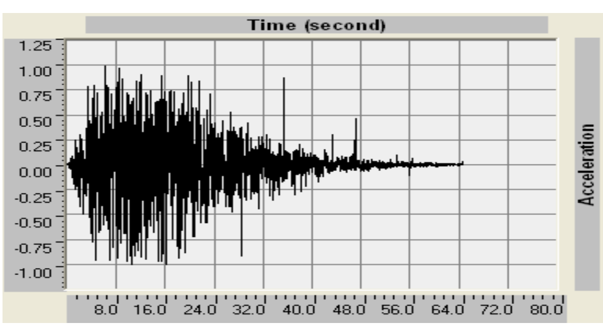


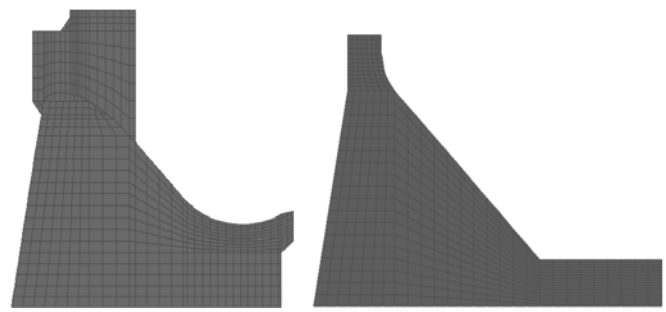
Fig. 2. Site-specific normalized time history of acceleration for Kameng project site.

FINITE ELEMENT MODELING

Geometry Modeling

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 was done separately in *SAP 2000*.

Based on the preliminary analysis performed with different sizes of foundation, the foundation size of $5b \times 2b$ was found to give acceptable response, where b is the base width of the dam. 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 Fig. 3.



(a) (b)

Fig. 3. Typical finite element mesh of (a) Overflow monolith (b) Non-overflow monolith with rigid foundation.

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 (γ_c) is taken as 24 kN/m^3, Poisson's ratio of concrete (ν_c) 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 is used in the analysis assuming the rock as homogeneous one. The mean rock properties are; Young's Modulus of Elasticity, E_f = 36410

MN/m^2, and Poisson's ratio of foundation rock (ν_f) = 0.33. The unit weight of foundation rock (γ_f) 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 (Chopra, 1967). It is represented as an assemblage of added mass based on this method. The maximum water level in the overflow section and non-overflow section are 67.25 m and 67.81m respectively. The unit weight of water is taken as 10 kN/m^3 and the velocity of pressure waves, C as 1438 m/s. To account the effect of reservoir bottom absorption, the wave reflection coefficient (α) given below is used;

$$\alpha = \frac{1-k}{1+k} \text{ ; where } k = \frac{\rho C}{\rho_r C_r} \qquad (2)$$

ρ = mass density of water

ρ_r = mass density of the foundation rock

C = Velocity of pressure waves in water

C_r = Velocity of pressure waves in foundation rock

For the chosen material properties, the wave reflection coefficient is found to be 0.74. 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.

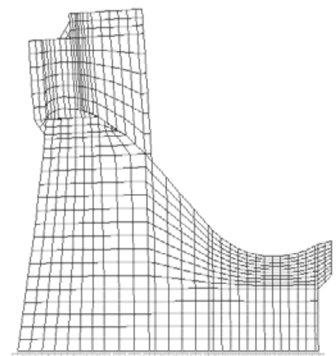
SEISMIC ANALYSIS

The seismic response of Bichom Dam in time domain was carried out for the estimated time-history of acceleration of Design Basis Earthquake (DBE) considering the site effects. Linear elastic behaviour is assumed. Seismic analysis was 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. The analyses were carried out for an empty and full reservoir condition. To investigate the influence of foundational flexibility effects, the dynamic response of the dam was performed assuming that the dam is founded on rigid foundation and flexible foundation. 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 were computed for different cases and are discussed below.

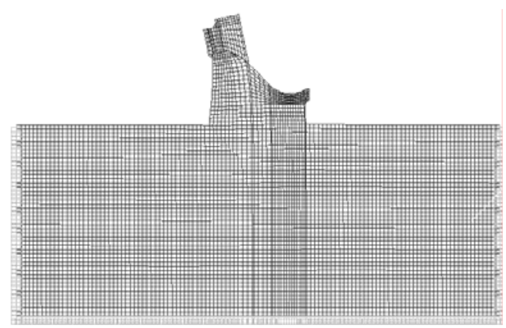
RESULTS AND DISCUSSION

Response of Overflow Monolith

The response of overflow monolith such as mode shape and time period, crest displacement, base reactions and stresses at the heel and toe of the dam obtained from the analysis is presented and discussed herein. The mode shape of overflow monolith at fundamental natural frequency for dam with empty reservoir condition is shown in Fig. 4. It is observed from this figure that the crest of the dam displaces in a same direction at fundamental mode for dam with rigid and flexible foundation and for empty reservoir condition. Similar trend is observed for full reservoir condition also.



(a) Rigid foundation



(b) Flexible foundation

Fig. 4. First mode shape for empty reservoir condition (a) Rigid foundation (b) Flexible foundation.

Time Period

The time period of overflow monolith for twelve modes and for the different cases: (a) Empty reservoir with Rigid foundation (E.R), (b) Full reservoir with Rigid foundation (F.R), (c) Empty reservoir with Flexible foundation (E.F) and (d) Full reservoir with Flexible foundation (F.F) is given in Fig. 5. It is found from the Fig. 5 that the trend of 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 fundamental time period obtained from the analysis of different cases is presented in Table 1. It is also observed from Table 1 that 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.

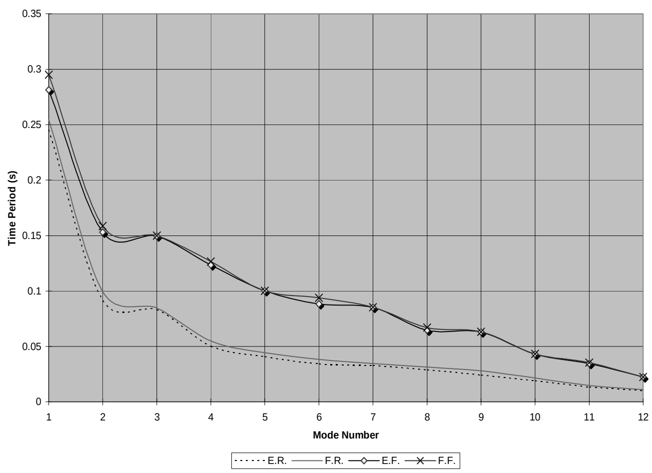


Fig. 5. Time periods for various cases of overflow monolith.

Crest Displacement

The typical time history of crest displacement obtained from the analysis for a dam with flexible foundation and empty reservoir condition is shown in Fig. 6. It is noted from the figure 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 cases is summarized in Table 1. It can be seen from Table 1 that for a dam with E.R, the crest displacement is 7.73 mm. It is also inferred from Table 1 that the crest displacement is substantially increasing (almost two times) when the foundation is flexible however the increase in crest displacement with full reservoir condition is only marginal.

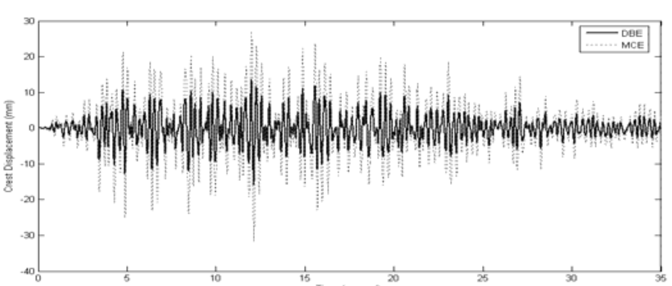


Fig. 6. Typical time history of crest displacement for dam with flexible foundation and empty reservoir

Base Reactions

The base shear and base moment obtained from the analysis for DBE are presented in Table 1. For the safety of dam, 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.

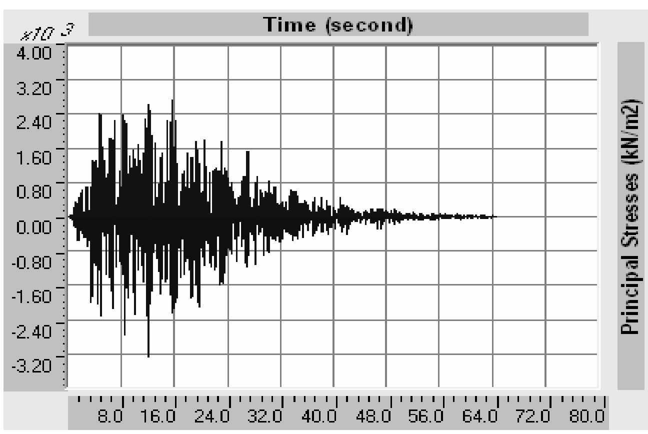
Table 1. Summary of Response of Overflow Monolith

Response		Rigid Foundation		Flexible Foundation	
		Empty Reservoir	**Full Reservoir**	**Empty Reservoir**	**Full Reservoir**
Time period (sec)		0.2453	0.2545	0.2814	0.2951
Crest Displacement (mm)		7.73	9.38	15.82	17.65
Base Shear (MN)		15.15	18.06	249.05	251.30
Base Moment (MN-m)		644.4	732.2	16944.6	17276.2
Max. Principal Stress (kN/m^2)	Heel	1271	1575	2734	3619
	Toe	824	893	1792	2174
Min. Principal Stress (kN/m^2)	Heel	1354	1604	3262	3632
	Toe	774	857	1881	2306

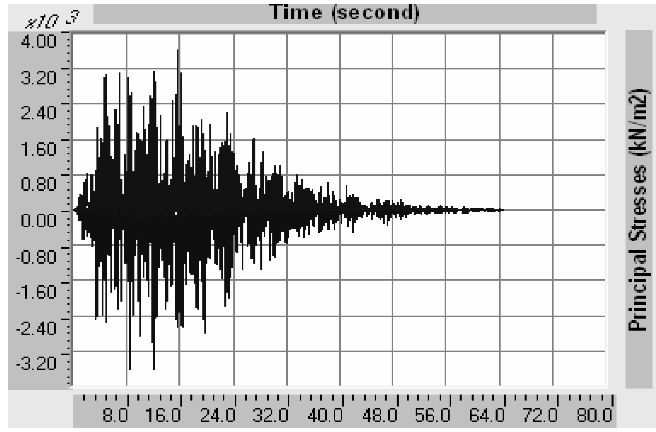
Stress Distribution

The concentration of stresses at the heel and toe of the dam were 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/mm^2 or 20000 kN/m^2. The tensile strength can be calculated as given by the Indian standard, IS 456 (2000), $f_{cr} = 0.7\sqrt{f_{ck}}$ N/mm^2; where f_{cr} is the characteristic strength of concrete in N/mm^2. 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 strength. Based on this criterion, the permissible tensile strength for DBE is estimated as 2500 kN/m^2 which is used in this study.

The time history of maximum principal stress (i.e. compressive stress) measured at heel off the overflow monolith with flexible foundation is shown in Fig. 7. The maximum and minimum principal stresses measured from the analysis for different cases are summarized in Table 1 for DBE excitation. Since 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 Fig. 7 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 strength 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 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.



(a) Empty Reservoir

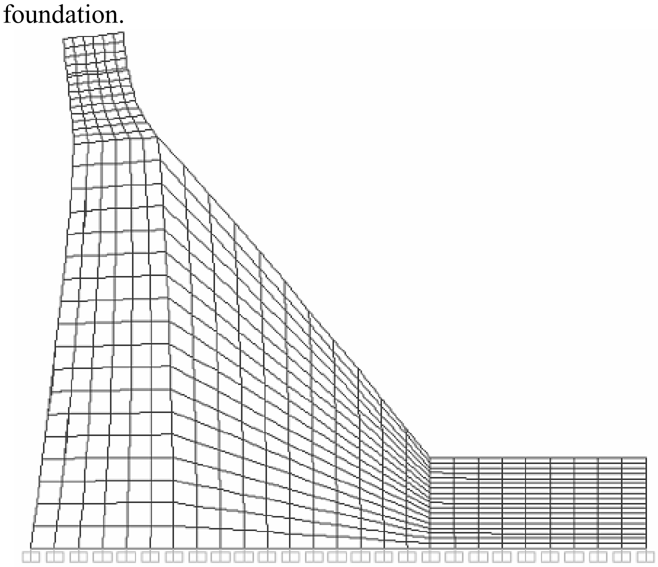


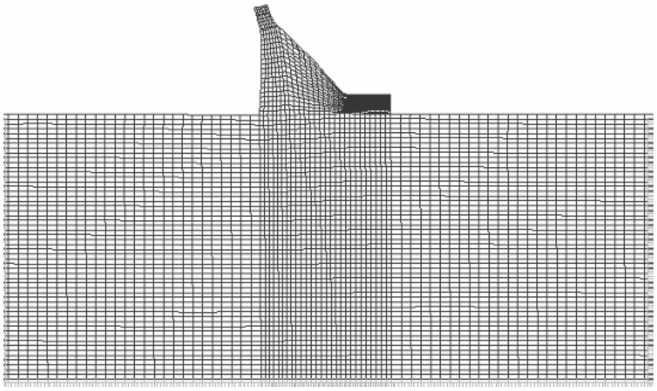
(b) Full Reservoir

Fig. 7. Time history of stresses at heel of overflow monolith with flexible foundation (a) Empty reservoir (b) Full reservoir.

Response of Non-Overflow Monolith

The response of 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 obtained from the analysis is presented and discussed herein. The mode shape of non-overflow monolith at fundamental natural frequency for dam with full reservoir condition is shown in Fig. 8. It is observed from the figure that the crest of the dam displaces in a same direction for dam with rigid and flexible foundation.


(a) Rigid foundation



(b) Flexible foundation

Fig. 8. First mode shape of non-overflow monolith with full reservoir condition.

Time Period

The time period of non-overflow monolith for twelve modes for the different cases similar to the one in overflow monolith is given in Fig. 9. It is found from the Fig. 9 that the time period of non-overflow monolith with mode number is also similar for both rigid and flexible foundation. It is also noticed from the figure that the time period of non-overflow monolith is also significantly influenced by the foundation flexibility and hydrodynamic effect of reservoir. However, it is found from the figure that hydrodynamic the effect on time period is significant at fundamental mode only, and only marginal at higher modes. The fundamental time period obtained from the analysis of different cases is presented in Table 2. It is observed from Table 2 that foundation flexibility increases the fundamental time period by 28 %, whereas the coupled foundation-reservoir interaction increases the fundamental time period by 50 %. This shows that the combined effect of foundation flexibility and full reservoir condition (hydrodynamic effect) is substantial on the fundamental time period of non-overflow monolith of dam.

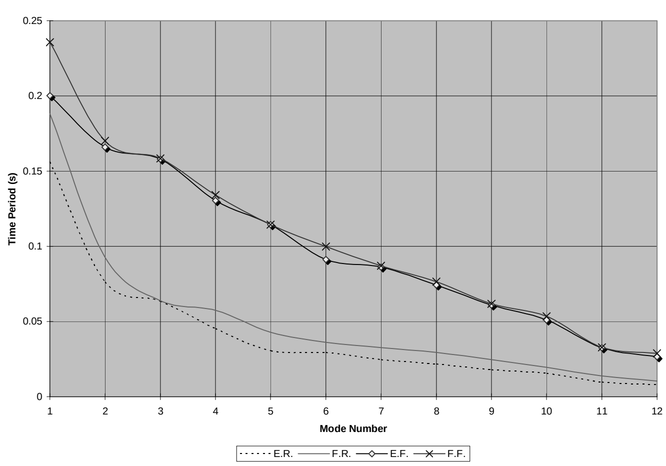


Fig. 9. Time Periods for various cases of non–overflow monolith.

Table 2. Summary of Response of Non-Overflow Monolith

Response		**Rigid Foundation**		**Flexible Foundation**	
		Empty Reservoir	**Full Reservoir**	**Empty Reservoir**	**Full Reservoir**
Time Period (sec)		0.1562	0.1883	0.2003	0.2357
Crest Displacement (mm)		4.73	6.89	13.33	17.55
Base Shear (MN)		10.90	17.03	330.62	335.98
Base Moment (MN-m)		299.1	459.5	25130.9	25140.7
Maximum Principal Stress (kN/m^2)	Heel	954	1592	3419	4490
	Toe	75	98	647	718
Minimum Principal Stress (kN/m^2)	Heel	876	1679	3358	4590
	Toe	120	96	664	715

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 monolith for an empty reservoir of rigid foundation is 0.2453 sec, while for non-overflow monolith is 0.1562 s. This indicates that the non-overflow monolith is comparatively stiffer than the overflow monolith. 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 with flexible foundation and empty reservoir condition is shown in Fig. 10. The maximum crest displacement measured from the time histories of displacement for different cases are tabulated in Table 2. From Tables 1─2, 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.

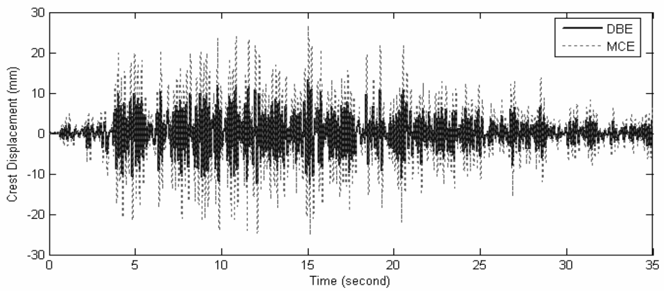


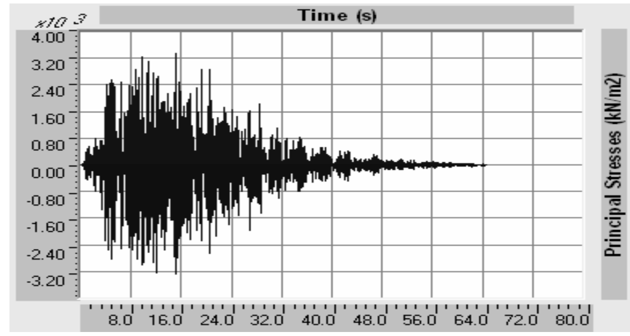
Fig. 10. Typical time history of crest displacement for dam with flexible foundation and empty reservoir.

Base Reactions

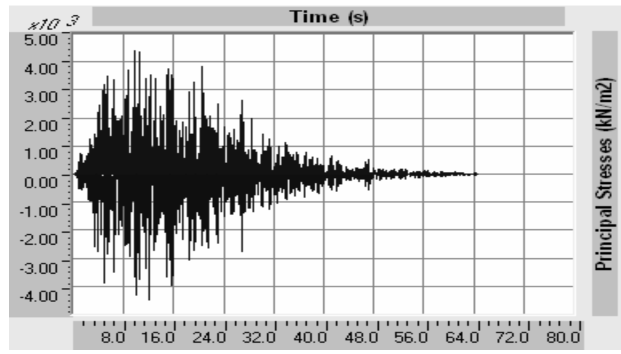
The base shear and base moment obtained from the analysis is given in Table 2. It is found from the table that the dam-water-foundation-interaction increases the base reactions. As discussed earlier, before overturning of the dam occurs, the cracks developing in the dam may lead to failure of dam due to exceeding stresses. Hence analysis of stress conditions is to be carried out to investigate the initiation of failure of dam cross section.

Stress Distribution

Since the grade of concrete used for overflow monolith and non-overflow monolith are same, the permissible strength of concrete under compression and tension are same as that of the values used for overflow monolith. The maximum and minimum principal stresses measured from the analysis for different cases are presented in Table 2. It is observed from Table 2 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/m^2 and 4590 kN/m^2 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/m^2 (permissible stress) all along the significant period and reaches the highest stress and reaching 3358 kN/m^2 at 15.12 sec after the excitation and reduces again below the limit (Fig. 11a). 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/m^2 lasting for about 25 sec during the excitation as shown in Fig. 11b. Therefore, this would result in extending the cracks along the section.



(a) Empty Reservoir



(b) Full Reservoir

Fig. 11. Time history of tensile stress in the non-overflow monolith with flexible foundation
(a) Empty reservoir (b) Full reservoir

CONCLUSIONS

The seismic behavior and performance of Bichom Dam when subjected to Design Basis Earthquake excitation was analyzed using a three-dimensional finite element model. The dam is modeled by solid elements consists of two cross sections viz. overflow monolith and non-overflow monolith. The analysis was done for four cases: (a) Empty reservoir with Rigid foundation (E.R), (b) Full reservoir with Rigid foundation (F.R), (c) Empty reservoir with Flexible foundation (E.F) and (d) Full reservoir with Flexible foundation (F.F). The conclusions arrived from the findings of the 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.

ACKNOWLEDGEMENT

Authors would like to thank North Eastern Electric Power Corporation Limited (NEEPCO), a Government of India Undertaking, for providing the required data for this study.

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