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Influence Of Modified Cross Section On Seismic Response Of Gravity Dam

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Abstract: Response of concrete gravity dam to earthquake is a complex phenomenon and reducing the effects of earthquake on these structures has always been an important concern .One such classic example is the Koyna Nagar Earthquake of 1967 which affected the functioning of Koyna Dam in India. This study evaluates the behavior of Koyna dam to different real ground motion accelerations and its implications by adopting dam-reservoir-foundation system. Response parameters such as displacement, acceleration, principal tensile and compressive stress at salient locations of the dam in response to past five major earthquakes ranging from low to high frequency content are investigated and compared. Development of tensile stress on the dam structure is the primary safety concern during an earth quake than compressive stress. Modified cross section is suggested to reduce the tensile stress at heel and slope changing points up to 25% and 35%respectively, the critical locations to avoid failure of the structure.

***Index Terms* - Dam-reservoir-foundation, PGA, Principal stress, Modified cross section.**

1.INTRODUCTION

The evaluation of dynamic behavior and seismic safety have been an area of interest for many researchers as failure of these structures may cause dangerous consequences. Dynamic behavior of a dam depends on the interactions among dam-reservoir and dam-reservoir-foundation systems. In gravity dam construction, joints are provided to separate monoliths. During an earthquake, inertia and hydro dynamic force acts both in horizontal and vertical direction. Hence the behavior of a dam is to be analyzed based components of both horizontal and vertical accelerations of the earthquake. Federal Guideline for dam safety[FEMA 2005] has established the excessive cracking during earthquake as the most important parameter while checking the safety of the dam. Any structural irregularities should be properly detailed to account for the localized effects of stress concentration. Severe cracking can lead to potential instability resulting from sliding and over turning. Sliding can affect an existing plane of weakness in the dam or foundation, or a plane of weakness formed by excessive cracking in the dam foundation interface. Hence design considerations play an important role in the case of gravity dams, for the structure to remain stable against disturbing loads through its geometric shape, mass and strength.

Fenves and Chopra [1984] developed a simplified procedure suitable for the preliminary design and safety evaluation of concrete dams. An equivalent single degree of freedom system was modelled which approximately represented the fundamental mode of concrete gravity dam. Bang- Fuh Chen [1996] studied in detail the nonlinear hydrodynamic effects of a concrete gravity dam on the safety aspects and suggested further modifications to the simplified method suggested by Fenves and Chopra [1984]. Bhattacharjee and Leger [1993] proposed a method that considered the dam reservoir foundation interaction using nonlinear analysis of concrete dams. Since the behavior of fluid and structure is expressed in terms of displacement, the equations of the motions of fluid system can be easily established through the Lagrangian approach which is used commonly for analysis and proved by A Caliyar& et al 1996,A Bayraktar & et al 1996 and M L Khan & et al 2009 . To investigate the influence of dam-foundation interaction on hydrodynamics, Saleh and Madabhushi [2010] used the dynamic centrifuge modeling technique. Løkke and Chopra [2014] presented response spectrum analysis approach for the design and safety evaluation of a concrete gravity dam. It is noticed that foundation has significant effect on the dam responses studied by R Sarkar & et al 2007,M Ghaemian& et al 2019 and H Mohammadnezhad & et al 2019 and the common mass less foundation approach overestimates the displacements and stresses within dam body proved by M Ghaemian & et al 2019 and H Mohammadnezhad & et al 2019 . This paper analyses the effect and implications of various large magnitude earthquake acceleration on a standard dam cross section. The Dam-reservoir-foundation system is implemented using finite element analysis software to understand the combined response of these systems to seismic acceleration. Analysis is also carried out on dam with modified cross section to understand the influence of variation in geometry on the response parameters.

2. DAM CROSS SECTION, MODELLING AND SEISMIC LOADING

2.1 Details of dam Cross Section

The Dam structure selected for the study is Koyna Dam which was affected by Koyna Nagar earth quake of 1967. The Koyna dam is103m in height with 70m wide base non over flow section. The dam is assumed to have a 350m x140m foundation as per Federal Energy Commission Guidelines [1999] for incorporating the effect of stress, based on structural dimensions. The extent of reservoir considered in the analysis is 140m in length, i.e. two times the width of the dam at the bottom. The cross section of the Koyna dam along with the reservoir and dimensions of foundation for the analysis is given in Fig.1.

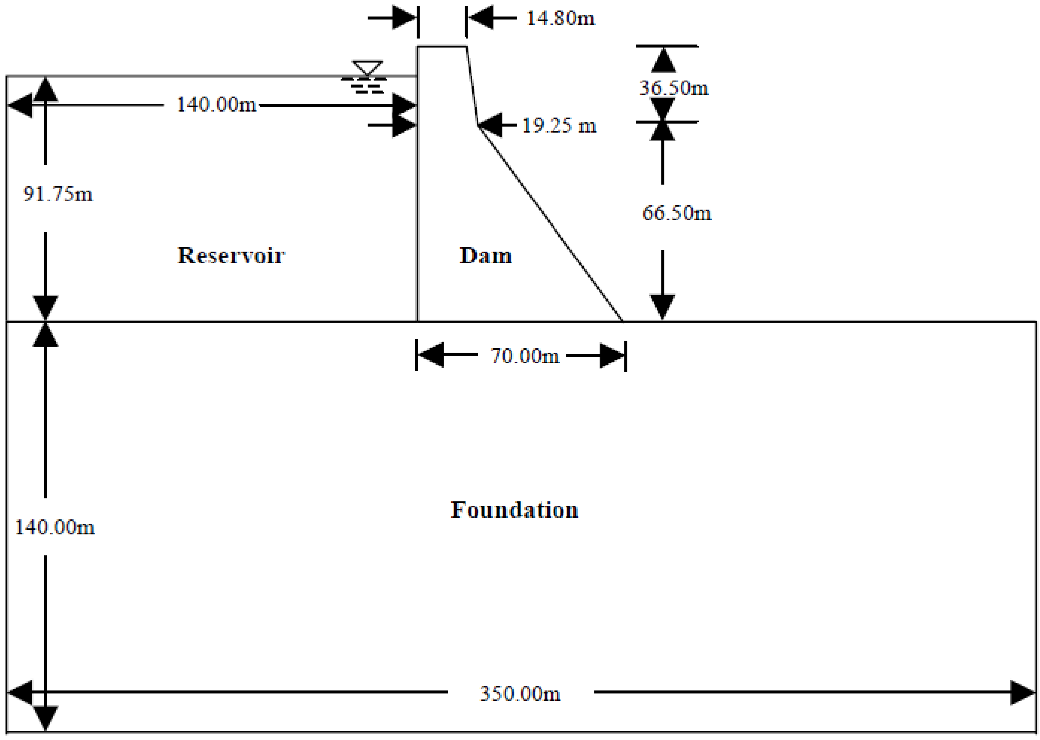


Fig. 1 Cross section of Koyna dam

2.2 Finite Element Implementation

By adopting Lagrangian approach, the complete finite element discretized equation for the fluid structure interaction (FSI) problem for the dam-reservoir-foundation system can be obtained by coupling the acoustic fluid and the structural matrices and can be represented as

$$\begin{bmatrix}[M_S] & [0]\\ \rho_0[R]^T & [M_F]\end{bmatrix}\begin{Bmatrix}\{\ddot{u}\}\\ \{\ddot{p}\}\end{Bmatrix} + \begin{bmatrix}[C_s] & [0]\\ [0] & [C_F]\end{bmatrix}\begin{Bmatrix}\{\dot{u}\}\\ \{\dot{p}\}\end{Bmatrix} + \begin{bmatrix}[K_S] & -[R]\\ [0] & [K_F]\end{bmatrix}\begin{Bmatrix}\{u\}\\ \{p\}\end{Bmatrix} = \begin{Bmatrix}f_S\\ f_F\end{Bmatrix} \qquad \text{Eq.1}$$

where$[M_S]$, $[C_S]$ and $[K_S]$ are the mass, damping and stiffness matrices of the structure (including dam and foundation) respectively and $[M_F]$, $[C_F]$ and $[K_F]$ are the mass, damping and stiffness matrices of the acoustic fluid. The fluid density is denoted by $\rho_0$and $[R]$ gives the coupling matrix which represents the coupling conditions on the interface between acoustic fluid and structure. $\{f_S\}$and$\{f_F\}$are structural and fluid load quantities produced at fluid structure interface in terms of unknown nodal displacement $\{u\}$ and pressure $\{p\}$.

Modeling and analysis of the dam is done using ANSYS16.Plane 182 element is used to model the dam and the foundation. The reservoir is modeled using Fluid 29 acoustic elements. The fluid structure interface are identified and fluid nodes are coupled at all interfaces with structure. The displacement and pressure degrees of freedom are solved simultaneously in the fluid-structure interaction model. The non over flow section is considered in the plane stress condition and foundation is considered as plane strain condition.

The material properties of concrete considered for the dam structure are [R Sarkar & et al 2007]: mass density 2643kg/m^3, modulus of elasticity 31027 MPa, tensile strength 2.9 N/mm^2, compressive strength 29 N/mm^2, and Poisson's ratio 0.2. Foundation is with material properties: density3300 kg/m^3, modulus of elasticity 62054 MPa and Poisson's ratio 0.33 and of reservoir water: density 1000 kg/m^3 , bulk modulus of 2250 MPa, and sonic velocity 1440 m/s. Self-weight of the structure and water pressure from the reservoir are the loads other than seismic loading considered in the analysis. All displacements are arrested at the bottom of the foundation.

2.3 Seismic Loading

Past five earthquakes of different characteristics are considered for evaluating the response of the dam to seismic loading. The particulars of the earthquakes are given in Table 1. As per the ratio of PGA in units of 'g' to PGV in units of 'm/s' (PGV), earthquakes are categorized as low frequency (PGA/PGV< 0.8), intermediate frequency (1.2<PGA/PGV< 0.8) and high frequency (PGA/PGV> 1.2) content [14]. Accordingly, the earthquakes considered for the analysis ranges from low frequency content to high frequency content. The moment magnitude of Park Field earthquake (2004) is the least though it is has the highest PGA with high frequency content among the earthquakes considered in the analysis.

Table 1 Details of past earthquakes considered for seismic analysis

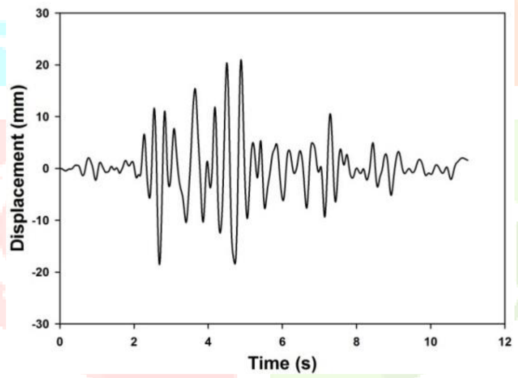
Name of Earthquake (Year)	Moment magnitude	Peak horizontal ground acceleration	Peak ground velocity (m/s)	Peak vertical ground acceleration	Category of earthquake based on frequency content
Koyna (1967)	6.5	0.4896 g	0.1965	0.245g	high frequency
Whittier (1987)	5.9	0.5371 g	0.2422	0.238g	high frequency
Park Field(2004)	5.5	0.9206 g	0.2568	0.458g	high frequency
Loma Prieta (1989)	7.0	0.6296 g	0.5519	0.478g	medium frequency
North Ridge(1994)	6.7	0.5839g	1.1449	0.283g	low frequency

3. RESULTS AND DISCUSSIONS

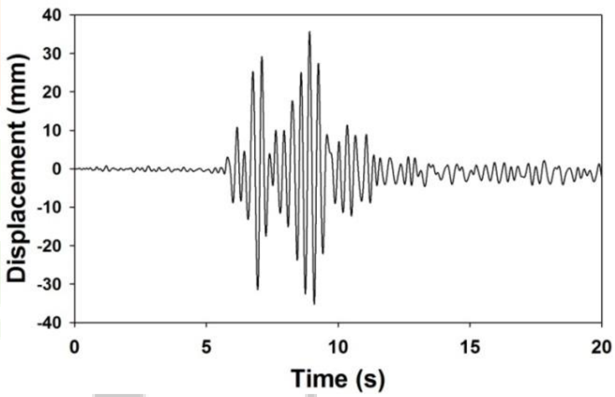
Time history analysis was carried out on the dam section with acceleration components of the past five earthquakes (Table 1)in horizontal and vertical directions simultaneously to understand the influence of the characteristics of the earthquakes such as moment magnitude, peak ground acceleration and frequency content on the response of the dam. The maximum response quantities like displacement, acceleration, normal vertical stress and principal stresses at salient sections of the dam were noticed. The influence of modified geometry on the response of principal tensile stress of the dam is compared with the original section.

3.1 Crest Displacement

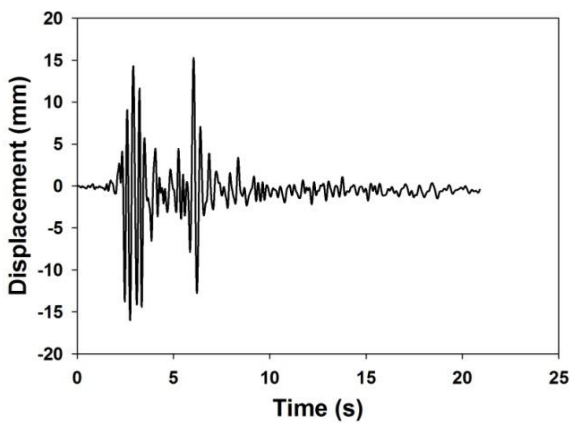
Time history of response displacement (horizontal) of the node located at crest under the action of various earthquakes is depicted in Fig. 2 (a-e). The time of occurrence of peak response has been observed between 2 and 9.04 seconds, depending on the time history of input motion.



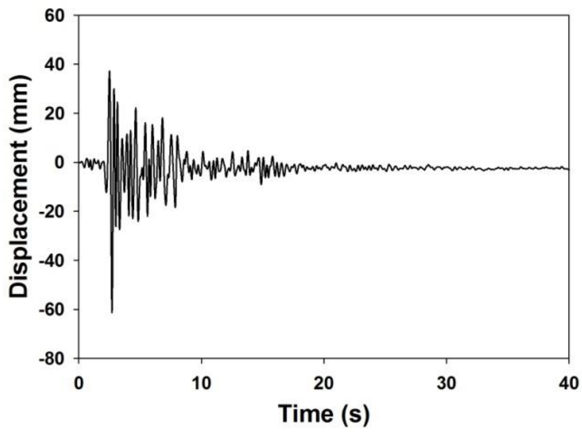
(a) Koyna Earthquake



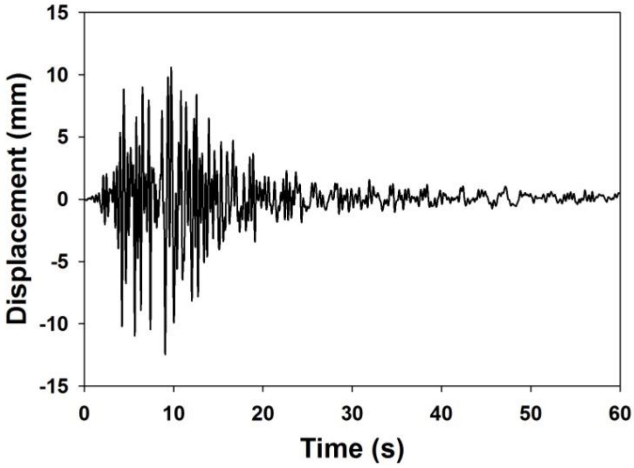
(b) Whitter Earthquake



(c)Park Field earthquake



(d) Loma Prieta earthquake



(e) North Ridge Earthquake

Fig. 2 Time history response of crest displacement under earthquakes

The peak values of crest displacement and heel displacement and its time of occurrence are also depicted in Fig.3. The highest crest displacement is noticed for Loma Prieta earthquake of medium frequency content with PGA of 0.6296 g. The higher displacement due to Loma Prieta earthquake may be attributed to its higher moment magnitude and impulsive nature of ground motion in the initial period. It may also be noticed that vertical acceleration of Loma Prieta earthquake is about 75% of the horizontal acceleration. Crest displacement due to Park field earthquake of PGA of 0.9206g has registered, only about 30% of that due to Loma Prieta earthquake, indicating that crest response depends on time history of input motion in addition to its PGA and frequency content. However, failure is not expected due to any of the seismic loading as the peak crest displacement is less than 1/1000 of the height of the structure and heel displacement is negligible (less than 5 mm) compared to the height of the structure.

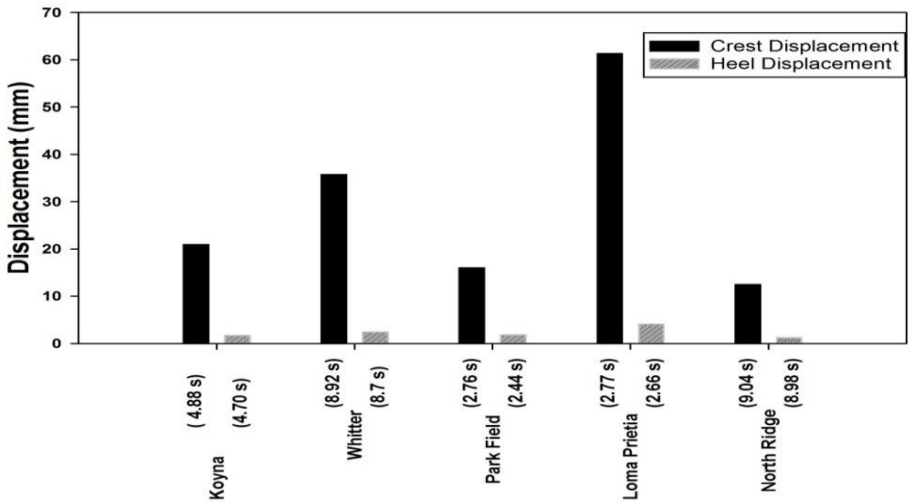


Fig. 3 Peak horizontal displacement at crest and heel due to earthquake loading

Time history of displacements of the node at crest, at Full Reservoir Level (FRL),slope changing point and heel corresponding to Koyna Earthquake are presented in Fig. 4, being the dam cross section considered in the present study is that of Koyna dam. The displacement of the structure increases along the height of the dam as expected. Maximum displacement observed at crest and slope changing points are less than 25 mm and 10 mm respectively.

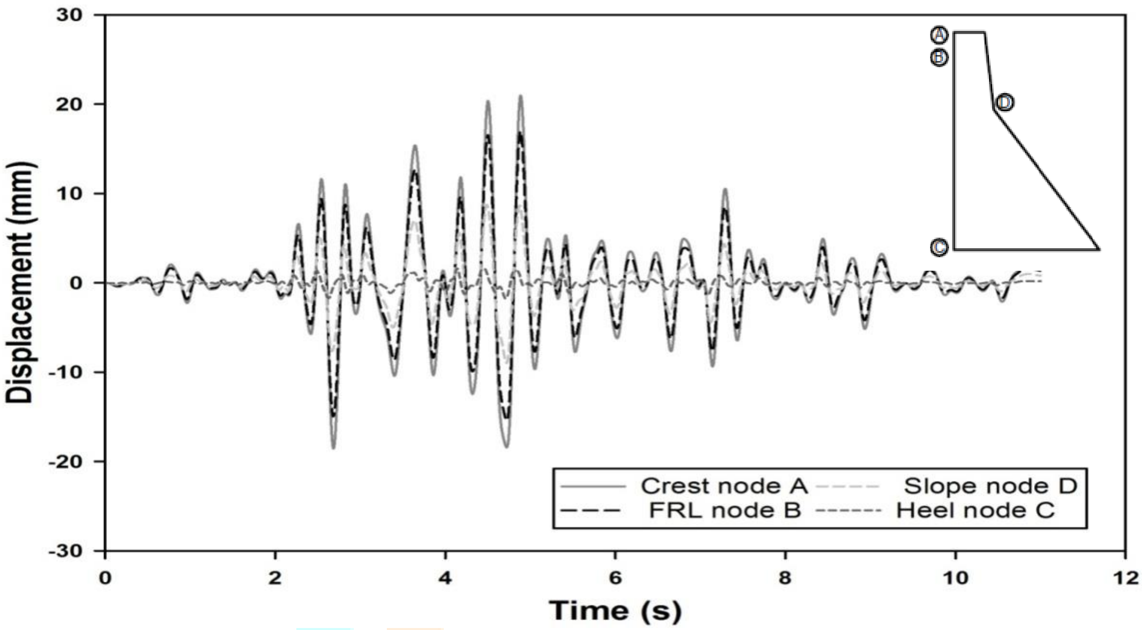
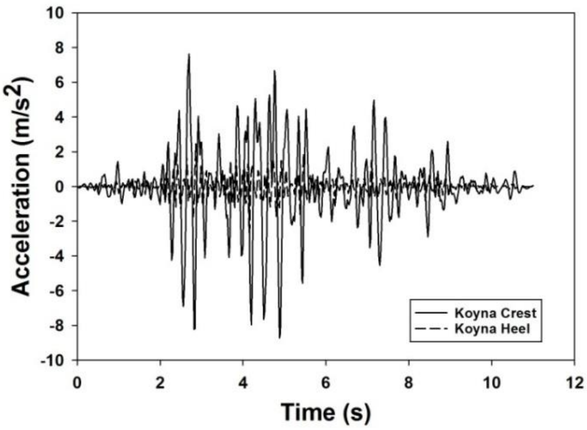
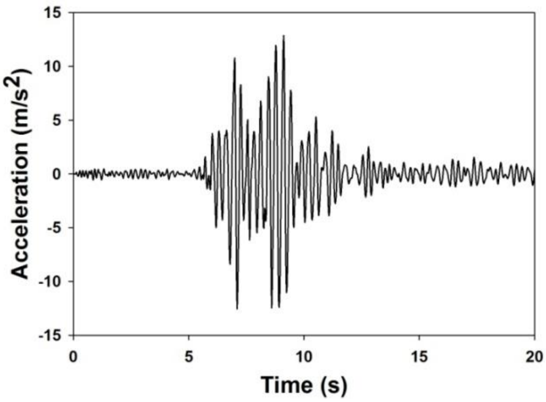


Fig. 4 Time history of response displacement at various heights of the dam (Koyna earthquake)

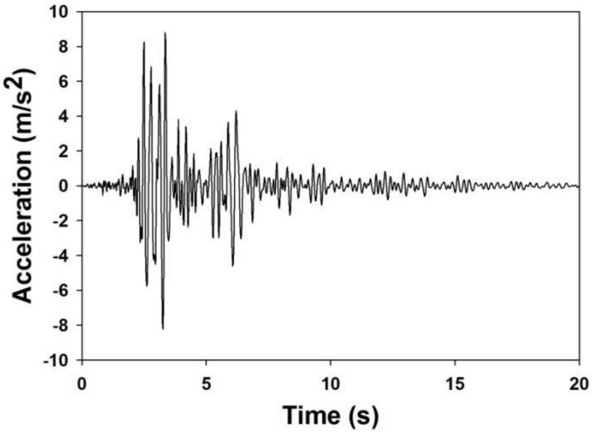
3.2 Crest Acceleration

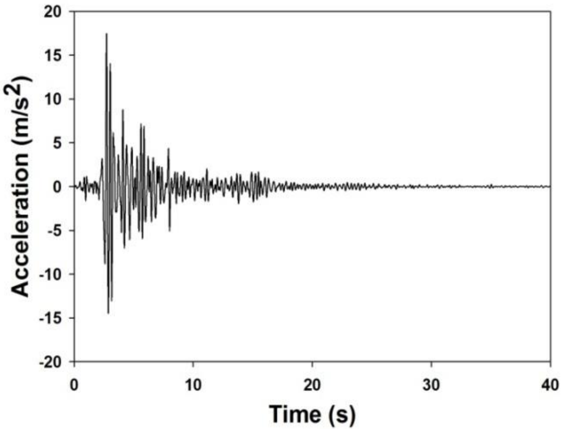
Time history of horizontal acceleration at crest under the action of earthquakes is depicted in Fig.5 (a -e). Fig. 5 (a) indicates same pattern of horizontal acceleration both at crest and heel due to Koyna earthquake inferring that monolithic action is maintained during the time domain of excitation. Peak acceleration noticed at crest and heel in all the five earthquake loadings are compared in Fig. 6 along with time of occurrence. As in the case of response displacement, the response acceleration at crest is the highest (17.5 m/s^2) for Loma Prieta earthquake and lowest for Northridge earthquake. Crest acceleration is observed to vary from 0.7 to 2.8 times of PGA of respective ground motion; lower value being attributed to the response to low frequency Northridge earthquake. However, acceleration at heel is always observed to be less than PGA of the corresponding earthquake.



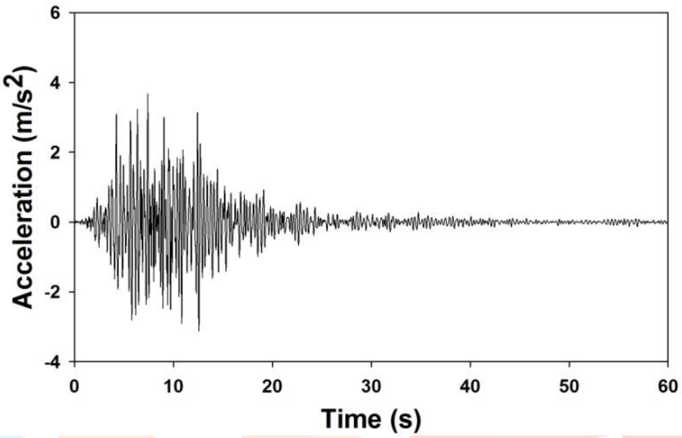


(a) Koyna earthquake (b) Whitter earthquake





(c)Park Field earthquake (d) LomaPrieta Earthquake



(e) North Ridge Earthquake

Fig.5 Time history response of crest acceleration under earthquake loading

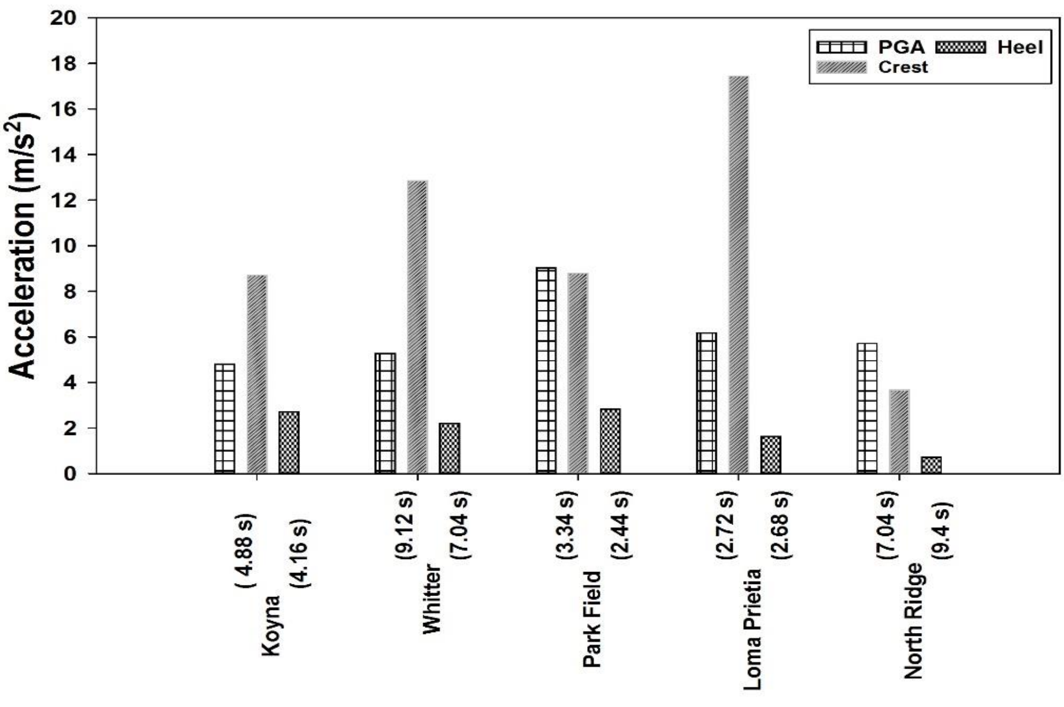
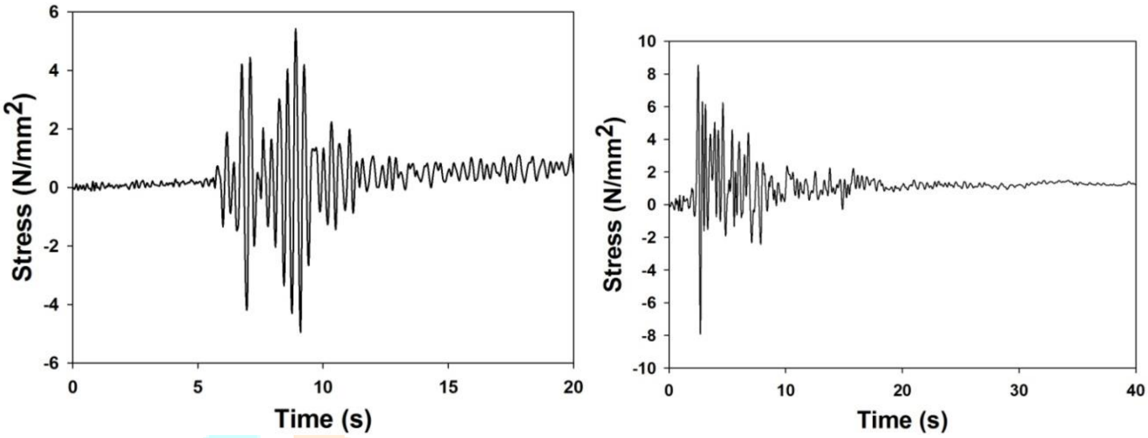


Fig. 6 Peak acceleration at crest and heel node

3.3 Analysis of Stresses

3.3.1 Normal Stress

It has been observed during the analysis that for all the ground excitations, maximum normal stress has occurred in vertical direction and that too at the heel of the structure. In all cases, peak response of vertical stresses are observed between 2 and 9 seconds as in the case of displacement/acceleration responses. Fig. 7 presents the vertical stress at the heel of dam under the transient loading of Whitter and Loma Prieta earthquakes which produced higher responses among the earthquakes in the present study.



(a) Whitter (b) Loma Prieta

Fig. 7 Time history response of vertical stress at heel due to earthquake loading

In order to check the criteria for the safety of structure from its tensile strength, time history of vertical stress at other salient points of the dam has been studied for various input motion. The peak vertical stress values at heel, slope changing point ,upstream side opposite to slope changing point and toe together with time of occurrence of peak stress are presented in Table 2. Similar to displacement and acceleration response parameters, higher value of tensile stress is also noticed in Loma Prieta earthquake indicating the significance of time history of ground motion on the response in addition to the PGA of earthquake.

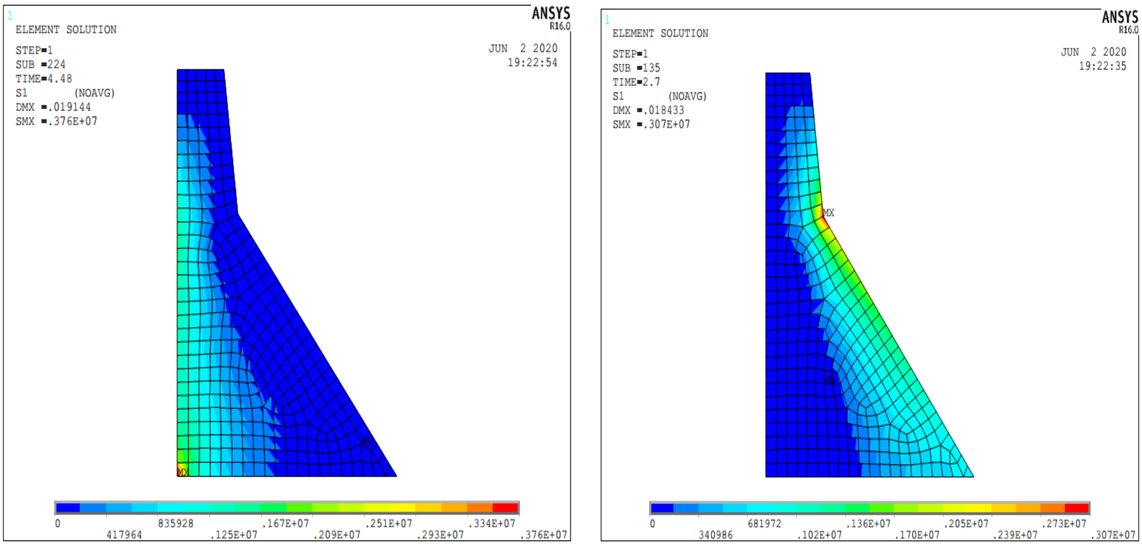
Table 2 Peak vertical tensile stress

Earthquake	Vertical stress (N/mm^2) and occurrence time (s)			
	at heel	at slope changing point	at upstream face; opposite to slope changing point	at toe
Koyna	3.28 (4.48 s)	2.40(2.70 s)	1.98 (4.90 s)	0.93 (4.72 s)
Whitter	5.41 (8.90 s)	4.11 (9.10 s)	3.54 (8.92 s)	1.438 (9.08 s)
Park Field	3.20 (3.22 s)	1.96 (3.36 s)	1.50 (2.92 s)	0.796 (2.44 s)
Loma Prieta	8.54 (2.5 s)	7.19 (2.72 s)	3.69 (2.88 s)	2.62 (2.68 s)
North Ridge	2.05 (9.70 s)	1.43(9.06 s)	0.99 (9.72 s)	0.573 (9.00 s)

In all the cases except North Ridge earthquake, tensile stress developed at heel exceeds the permissible tensile stress (2.9 N/mm^2). However, dynamic tensile strength which is considered as twice the permissible tensile stress[15] of concrete is more than the peak tensile stress at heel except in the response of the Loma Prieta earthquake. Results indicate the initiation of cracks and its propagation from the heel towards the toe and reduction in stiffness of the structure. This tensile cracking can cause sliding and rocking of the dam with a permanent offset. At the slope changing point, the likely point of stress concentration, tensile stress developed exceeds the permissible values in the case of Loma Prieta, and Whitter acceleration data. At the upstream face, opposite to the slope changing point similar tensile stress failure can be noticed for these two earthquake loadings. However at the toe, none of the ground motions cause failure conditions. This indicates the necessity of increased base width along with provisions for steel reinforcement and introduction of fillet in upstream faces to reduce the tensile stress at heel.

3.3.2MaximumTensile Stress and demand capacity ratio

Maximum value of major principal stress, indicating the maximum value of tensile stress under the action of different earthquakes has been noted. Fig.8depicts the contour of the principal tensile stress upon the action of Koyna earthquake at the instant of maximum principal stress (a) at the heel of the dam and (b) at the slope changing point. Though principal tensile stress developed at heel is much higher than the permissible stress, the stresses developed on the sections away from the vicinity of the heel are within the permissible limit. The maximum value of the principal stress which occurred at the heel, the slope changing point and the toe of the dam in all cases are compared in Fig. 9along with the static and dynamic tensile strength of concrete.



(a) maximum stress at heel (b) maximum stress at slope changing point

Fig. 8 Principal tensile stress due to Koyna Earthquake

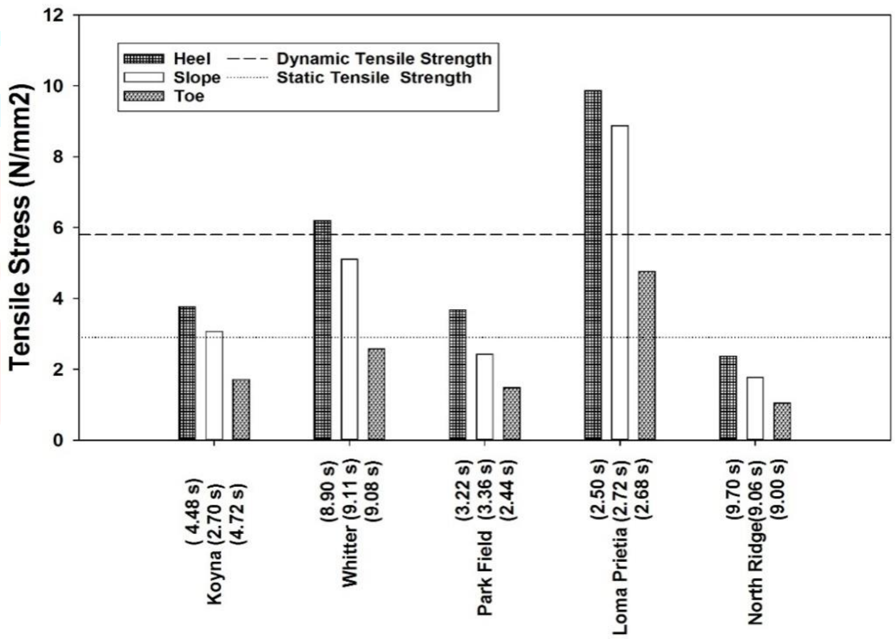
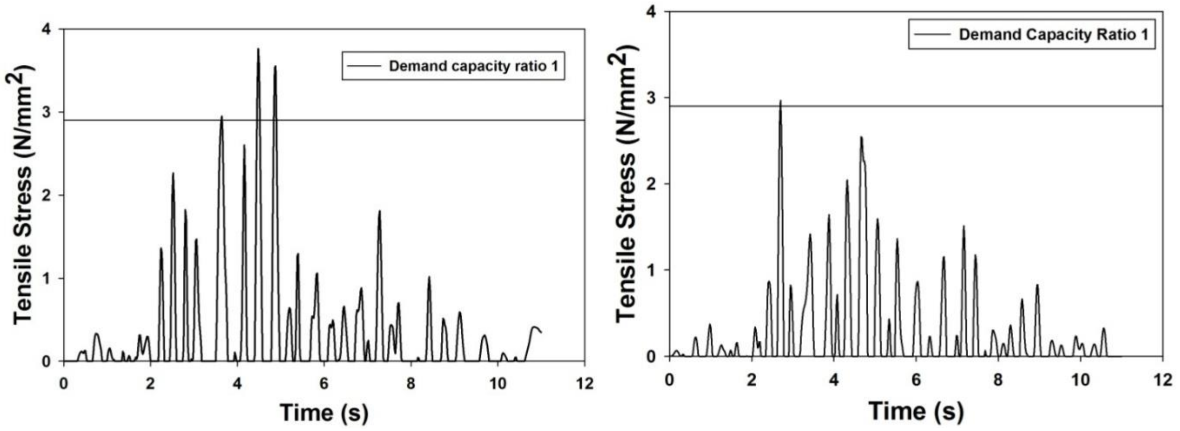


Fig. 9 Peak values of principal tensilestress

Principal stress valuesexhibitsimilar variations as that of vertical stress at theheel, the slope changing point and at the toe in all the cases of loading as expected. Principal tensile stress exceeds the strength of concrete at the heel in all loading cases exceptin the case of lowfrequency Northridge earthquake. Stress developed due to earthquake excitation of Loma Prieta and Whitterare respectively 2.6 and 1.6 times that of the stress caused by the Koyna earthquake. It can be noticed that tensile stress at slope changing point varied from 65% to 90% at the heel, suggesting the importance of attention to this critical point.However, no tension failure is expected at toe as the tensile stress at the toe in all cases of response is less than the strength of concrete.

Since the maximum tensile stress atthe heel and the slope changing points have exceeded the strength of concrete, time history of principal tensile stress is checked against the demand capacity ratio of 1.0 through Fig. 10 for loading of Koyna earthquake.Tensile stress at heel is observed to exceed the capacity for about 2 s. At the slope changing point tensile stress is less than the capacity for most of the duration and any increase in the stress exceeds the capacity only by a very small margin. Thoughthe tensile stress pulses are isolated and do not represent a continuous series of excursion beyond the tensile strength threshold, it may cause tensile cracks.The dam would exhibit nonlinear responses in the form of cracking of the concrete or opening of the construction joint as the estimated stress demand capacity ratio exceeds one. However, the response is considered acceptable as demand capacity ratios less than 2.0 and the overstressed region is below 15% of dam cross sectional area in line with US army corps of Engineers[2003,2007]



(a) at heel (b) at slope changing point

Fig. 10 Time history of principal tensile stress at heel and slope due to Koyna earthquake

3.3.3MaximumCompressive Stress

Maximum principal compressive stresses at the heel of the dam for transient loading of various acceleration data are presented in Fig. 11. In all the cases of time histories,compressive stress has not exceeded the compressive strength of concrete. Maximum compressive stress induced in the structure is less than 10 N/mm2. This indicates that the compressive failure cannot be expected in the dam due to seismic loadings of the type considered in the present study.

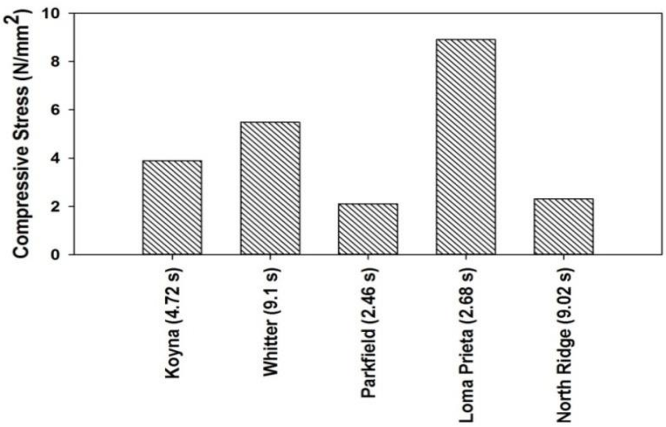
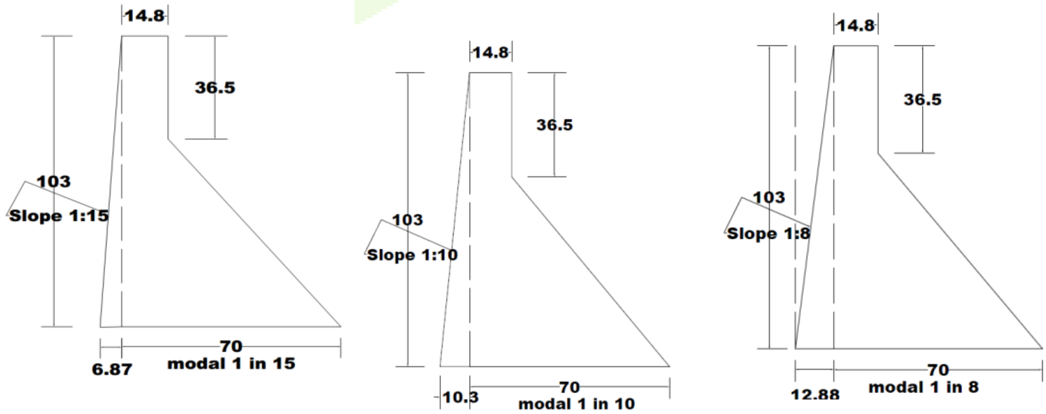


Fig. 11 Peak values of principal compressive stressat heel

3.4 Modified Cross section

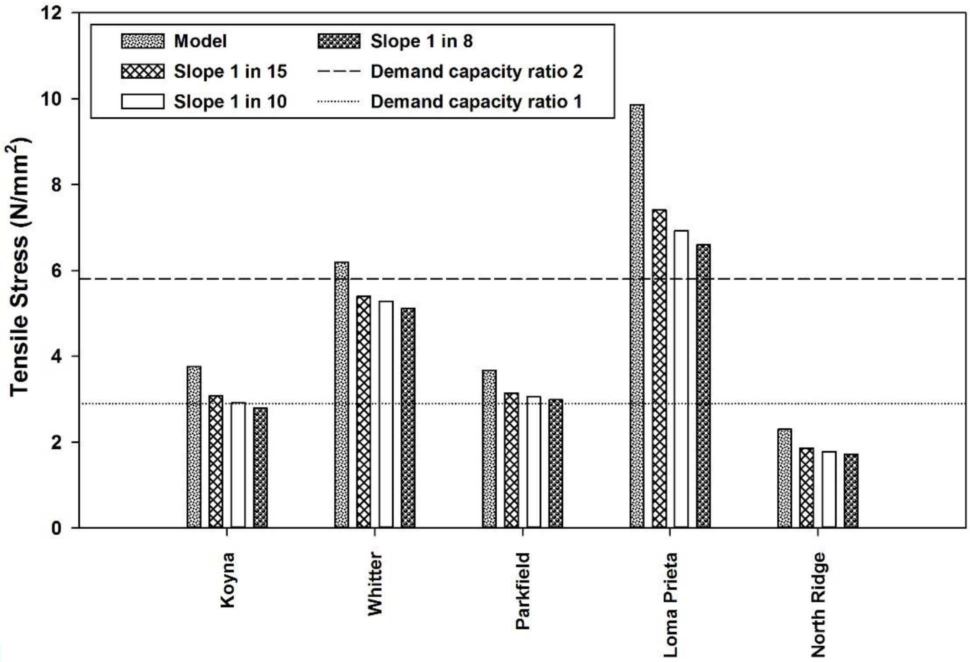
In order to reduce the effect of tensile cracking,dam section is modified by giving slope towards theupstream side. The horizontal to vertical (H:V) ratios adopted are (i) 1 in 15, (ii) 1in 10 and (iii) 1 in 8 as shown in Fig. 12.



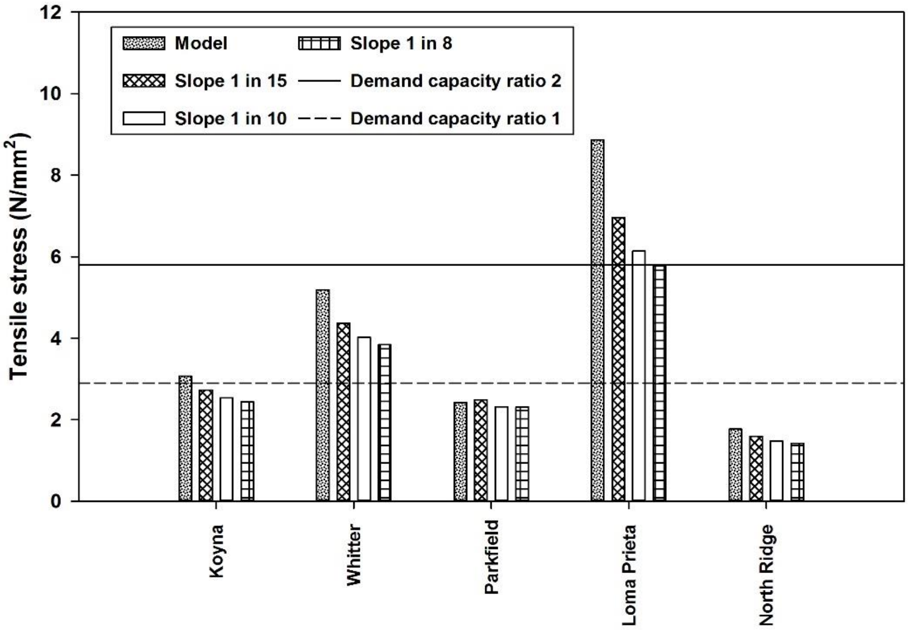
(a) H:V; 1 in 15 (b) H:V; 1 in 10 (c)H:V;1 in 8

Fig. 12 Modified geometry of the dam

The maximum principal tensile stress at the heel and the slope changing point of the original section is compared with the modified geometryin Fig.13. It can be noticed that upstream fillet in the dam body can reduce the heel tensile stressfrom 18to25%for H:Vratio from 1 in 15 to 1 in 8. For the seismic loading of Koyna, ratio of 1 in 10 or more is required to bring down the tensile stress, belowdemand capacity ratio 1 atthe heel. Fig. 13 (b) indicates that upstream fillet also reduces the stress at slope changing point. Upto 35% reduction in stress can be noticed in loading due to Loma Prieta earthquakewhich registered the highest response. Slope of 1 in 8 reduces the tensile stress to a level less than demand capacity ratio of 2 in all cases at slope changing point of the section.



(a) atthe heel



(b) at the slope changing point

Fig. 13 Peak values of principal tensile stress for modified cross section

4. CONCLUSION

Time history analysis of past five earthquakes namely Koyna (1967), Whittier (1987), Loma Prieta (1989) and Northridge (1994) and Park Field (2004) on Koyna dam section infers that the various responses of a dam structure to earthquake excitation depend on the time history of input motion in addition to the PGA and frequency content of the earthquake. The conclusions drawn from the present study are given below.

1. Horizontal acceleration at crest of the dam varies from 0.5 to 2.7 times that of PGA of respective ground motion.

2. Mximum tensile stress exceeds the tensile strength of concrete at heel in all cases of seismic loading except loading due to lowfrequency Northridge earthquake, suggesting the requirement of steel reinforcement at heel.
3. Demand capacity ratio more than 2 is registered in the case of Loma Prieta earthquake of medium frequency content and Whittier earthquake of high frequency content.
4. Stress developed due to seismic excitation of Loma Prieta and Whitterare respectively 2.6 and 1.6 times that ofKoyna earthquake.
5. Maximum tensile stress observed at slope changing point is about 90% of that at heel.Time history of principal tensile stress at slope changing point implies the requirement of due modification of cross section and tensile strengthening with reinforcement.
6. Section is safe forcompressive strength and crushing of concrete is not expectedin concrete of strength 29 MP in any of the ground motions studied.
7. No failure is expected at the toe either in tension or compression, hence suggesting upstream fillet than downstream for reducing the stresses.
8. Upstream fillet in the dam body can reduce heel tensile stress from18to 25% for fillets of H: V ratios from 1 in 15 to 1 in 8 and stress at slope changing point up to 35%.

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