Numerical analysis of RCC dam in the high seismic zones of higher Himalayas

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ABSTRACT:

A concrete gravity dam with a maximum height of 160 m is proposed for the Phukot Karnali Hydroelectric Project (PKHEP) located in the Karnali Province of Nepal. The design of dam is based on US-Army Engineering Manuals. Five sets of response spectrum matched time histories obtained from deterministic seismic hazard analysis (DSHA) are applied for liner-elastic time-history analysis of the dam. The results obtained from linear-elastic time-history analysis are compared with the EM 1110-2-6051 performance acceptance criteria for gravity dam. The study indicated that the dam response parameters are within acceptable limit except at very narrow area of the dam heel. Further details regarding the failure modes are to be analyzed using non-linear time history analysis. The present result also shows that the dam geometry can further be optimized by performing non-liner time history analysis.

Key words: Gravity dam, linear time-history analysis

1 INTRODUCTION

Nepal has a large potential of hydropower resources. The rapidly changing topography and consequently, the high river gradient provides some of the ideal location for hydropower development. However, most of this potential remains untapped.

The combined effort from the government and the independent power producers (IPPs) since the start of this century has brought about significant changes. Nearly 2500 MW of hydropower projects are under construction and number of large hydropower projects are in planning and study phase. The projects under construction includes two major dam projects namely, 900 MW Arun III (70 m high dam) and 140 MW Tanahu Hydropower Project (140 m dam). The remaining projects are run of river projects with small diversion dam. Government of Nepal is planning several high dams to meet increasing demand by developing peaking and storage projects.

Lying between the Indian and Eurasian continental plates, Nepal is in one of the most tectonically active part of the world. The main challenges associated with high seismicity of Higher Himalayas is the stability of dam and appurtenant structures during seismic event and management of risk at downstream. Following type design or simpler design methodology allow for large conservatism that ultimately leads to an expensive design, thus a more detailed analysis that takes into account the site-specific seismic risk is necessary for better assessment of downstream risk as well as a more optimized design.

This paper summarizes the earthquake response evaluation of Phukot Karnali hydroelectric project (PKHEP) dam by using linear elastic time-history analysis.

2 THE PHUKOT KARNALI HYDROELECTRIC PROJECT

The PKHEP is located in Kalikot district, Karnali Province of Nepal. The project plans to use the flow from the Karnali River to generate 480 MW of electricity. The proposed headworks of the project is about 1.5 km downstream from the confluence of Karnali and Sanigad River. The main civil structures of the project are roller compacted concrete (RCC) dam of maximum height 160 m, intake, upstream (U/S) and downstream (D/S) coffer dams, two diversion tunnels, two headrace tunnels of length about 6 km, two surge tunnels, two pressure shafts/tunnels, underground powerhouse cavern, underground transformer cavern, and two tailrace tunnels. Figure 1 shows the general layout plan of PKHEP dam.

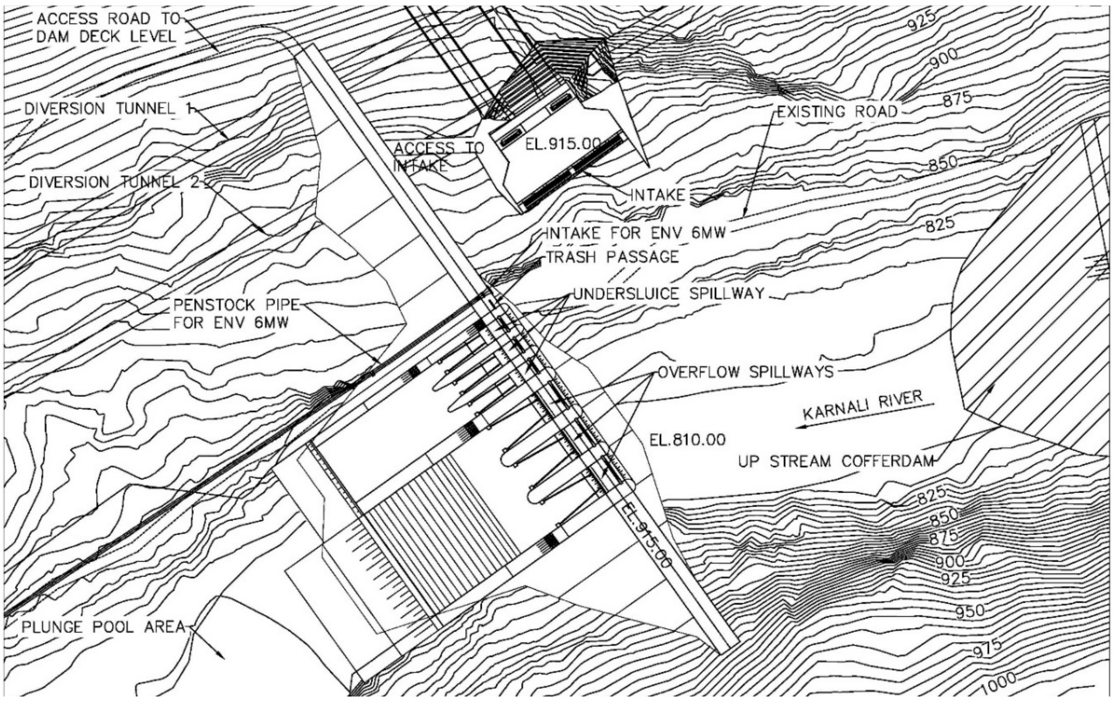

Figure 1 General layout of Phukot Karnali HEP Headwork Plan

3 SEISMIC HAZARD IN HIGHER HIMALAYAS

The higher Himalayas in Nepal is the part of Himalayan arc that extends about 2400 km between the Namcha Barwa (Tibet) and Nanga Parbat (India). This immense range was formed by tectonic movements and sculpted by weathering and erosion. The higher Himalayas in Nepal form the central segment of the complex Himalayan arc. The Himalayan range can be subdivided into several tectonic units that extends throughout its length. These units include the Siwalik Hills, Lesser Himalayas, Higher Himalayas and Tibetian Tethys. These zones are bounded by four major thrust sheets, namely the Main Frontal Thrust (MFT), Main Boundary Thrust (MBT), Main Central Thrust (MCT) and South Tibetian Detachment System (STDS) (Pandey at al 1999). These nearly vertical thrust sheets (MFT, MBT, MCT) meet the sub horizontal thrust layer (MHT) on which the southern Indian content subdues beneath the Eurasian content. Together these thrust systems can produce an earthquake of over 8.0 magnitude. The Phukot Karnali HEP lies in a klippe of higher-grade rocks and is bounded by MCT in all directions.

Due to very high seismicity of the region and nearness of seismic sources, any dams constructed in Higher Himalayas could be subjected to near field ground motions from massive earthquakes. The information of ground motion in near-field region of damaging earthquake is limited due to the sparse seismic network (Acharya et al. 2019). The character of near-field ground motions may differ significantly from that of far-field ground motions. Few great earthquakes have been recorded in the recent past, but these databases are not enough to develop the ground motion prediction equations (GMPEs).

To determine the design ground motion at PKHEP dam location, seismic hazard analysis of the project site was performed following the guidelines of ICOLD 148. The PSHA study recommended an OBE of 0.09g. Similarly, the DSHA study has estimated PGA 0.41g for SEE considering a Mw 7.8 event along the MHT. The results of the DSHA have been used in analysis.

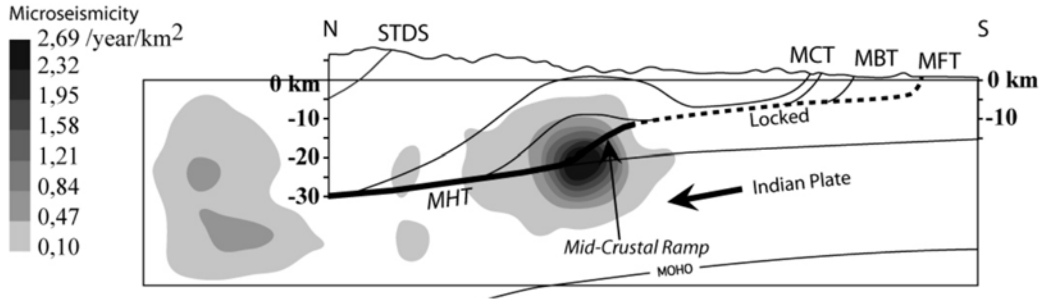

Figure 2 North-South geological cross-section of Nepal Himalaya showing major thrust system and micro-seismic activity (Pandey et al., 1999)

4 MATERIAL PROPERTIES

The properties of roller compacted concrete (RCC) and foundation rock used for design are shown in Table 1 and Table 2.

Table 1 Properties of Roller Compacted Concrete

Unit weight	2400 kg/m^3
Unconfined static compressive strength, f'_c	20 MPa
Young's Modulus	21000 MPa
Poisson ratio	0.2

Table 2 Properties of foundation rock

Rock type	Augen Gneiss
Young's Modulus	25000 MPa
Poisson ratio	0.2
Friction coefficient	0.9
Cohesion, C	0.8 MPa

5 STABILITY ANALYSIS

Before finite element analysis of the proposed dam, stability analysis of tallest section, overflow spillway section (Figure 3), was performed as per the load combinations described in "EM 1110-2-2200, Gravity Dam Design". Table 3 shows summary of 2D stability analysis of overflow section of dam. Conventional stability analysis is performed as per EM 1110-2-2200 for seven different load combinations: (i) construction, (ii) normal operating, (iii) flood discharge, (iv) construction with OBE, (v) normal operating with OBE, (vi) normal operating with MCE and (vii) probable maximum flood. Dam geometry obtained from 2D stability analysis is used as the initial trial section for FEM analysis.

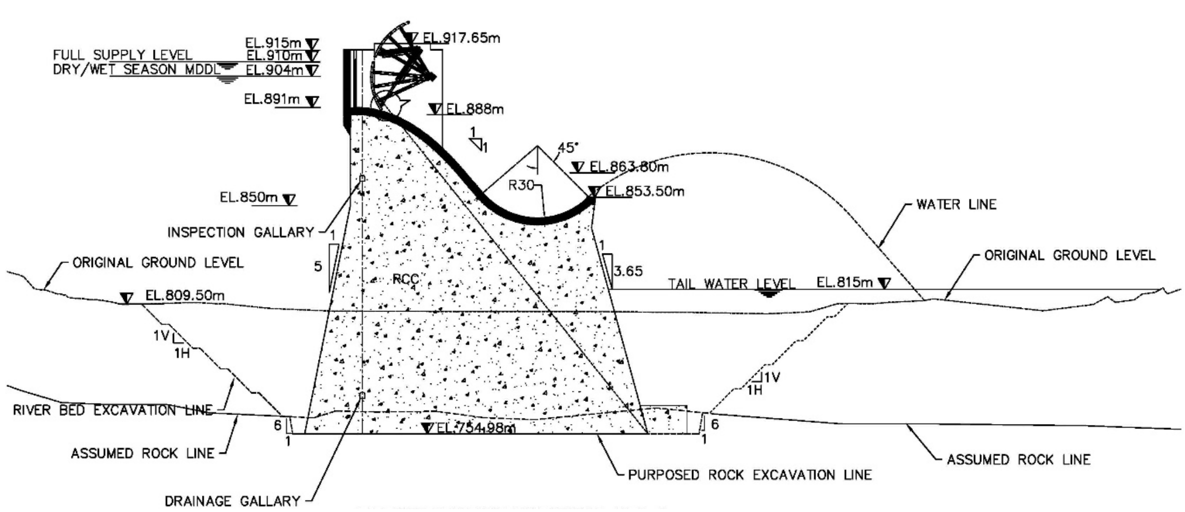

Figure 3 Dam Overflow Section

Table 3 Stability Analysis of dam overflow section

	LC1 Unusual	LC2 Usual	LC3 Unusual	LC4 Extreme	LC5 Unusual	LC6 Extreme	LC7 Extreme
Eccentricity,	-5.43, Within Middle 1/2, SAFE	21.56, Within Middle 1/3, SAFE	21.52, Within Middle 1/2, SAFE	8.6, Within Base, SAFE	28.29, Within Middle 1/2, SAFE	45.03, Within Base, SAFE	23.22, Within Base, SAFE
P_{max}	1.69, < 3.4, SAFE	2.97, < 3.4, SAFE	2.9, < 3.4, SAFE	2.98, < 4.52, SAFE	3.41, Redesign	4.51 < 4.52, SAFE	3, < 4.52, SAFE
P_{min}	2.69, < 3.4, SAFE	0.15, < 3.4, SAFE	0.15, < 3.4, SAFE	1.4, < 4.52, SAFE	-0.29, Tension-Check Crack Length	-1.39, Tension-Check Crack Length	0.04, < 4.52, SAFE
FOS, Sliding	Infinity, SAFE	2.8, SAFE	2.82, SAFE	20.54, SAFE	2.26, SAFE	**1.36, SAFE**	2.7, SAFE
FOS, Uplift	Infinity, SAFE	3.04, SAFE	2.9, SAFE	Infinity, SAFE	3.04, SAFE	3.04, SAFE	2.88, SAFE

6 FINITE ELEMENT ANALYSIS

In the finite element model, the dam and foundation rock are represented by 2D plane strain elements of unit thickness. For the RCC, a modulus of elasticity of 21000 MPa, a poison’s ratio of 0.2, with a unit weight of 2400 kg/m3 is assumed. The foundation rock is assumed as being massless and its modulus and Poisson’s ratio were assumed as 250000 MPa and 0.2 respectively. The finite element analysis is performed to determine the in-plane response of the critical section, which corresponds to the tallest overflow monolith. The height of the monolith is 136 m, and the base is 143 m. Figure 4 shows the geometry and the finite element mesh used for the analysis. The inertial forces of the impounded water are represented by added mass at the associated nodal points. A total of 30,568 shell elements are used in analysis of which 3368 elements represent the dam section and the remaining represent the foundation rock. The model includes a total number of 30,966 nodal points. Length of the foundation model is 543 m and depth are 200 m which satisfies the criteria specified in EM 1110-2-6051 and EM 1110-2-6053.

Figure 4 FEM model of dam and foundation

6.1 *Earthquake Ground Motion*

Five sets of earthquake ground motions (Figure 5) are considered for the analysis, all of them scaled up to the same values of the peak acceleration based on site specific seismic hazard analysis.

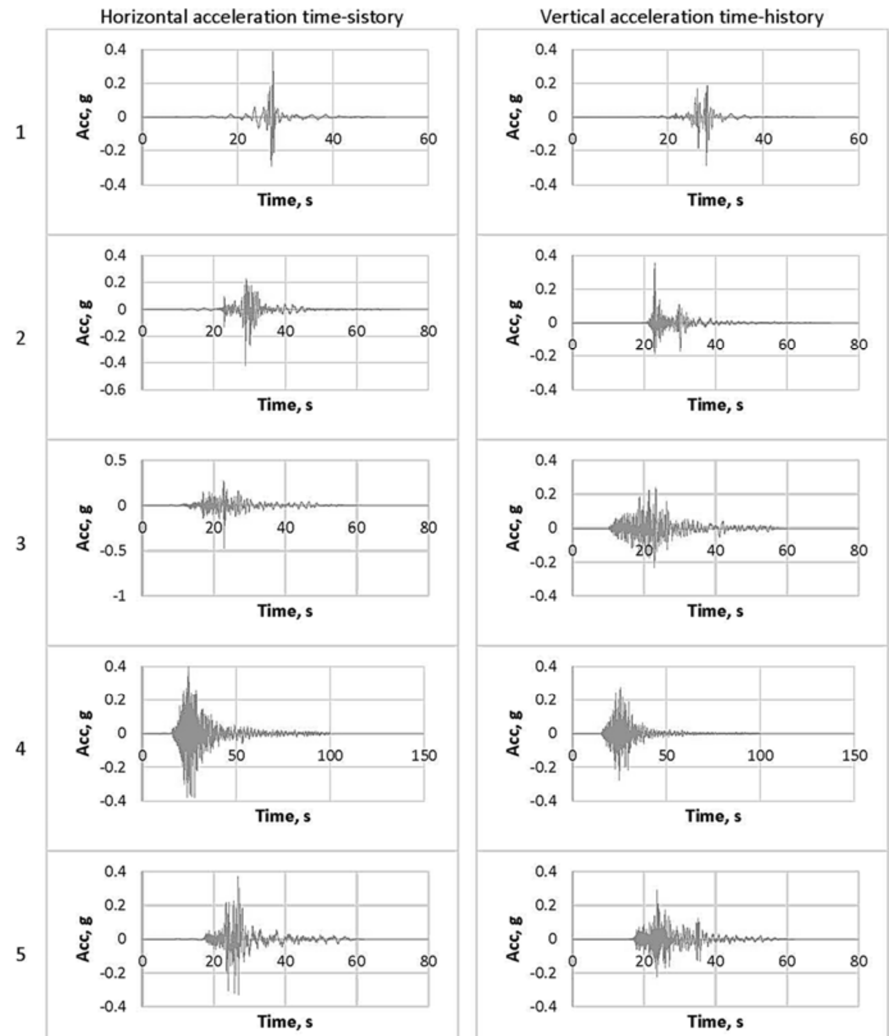

Figure 5 Five sets of earthquake ground motions with maximum PGA 0.41g

6.2 *Dynamic characteristics*

The first ten natural frequencies corresponding to both empty and full reservoir conditions are shown in Table 4. When compared to the empty case, it can be noted that in general the frequencies are reduced by about 10 % in the presence of a reservoir, except for those corresponding to the second and fifth mode shapes, which are less sensitive.

Table 4 Modal period and frequencies

Mode	Empty		Full	
	Period, Sec	Frequency, Cyc/sec	Period, Sec	Frequency, Cyc/sec
1	0.46	2.18	0.53	1.88
2	0.24	4.1	0.24	4.09
3	0.19	5.32	0.21	4.84
4	0.11	8.91	0.12	8.07
5	0.09	10.98	0.1	10.38
6	0.08	13.01	0.09	11.25
7	0.06	15.69	0.08	13.18
8	0.06	16.52	0.07	15.33
9	0.04	24.55	0.05	20.71
10	0.03	28.64	0.04	24.93

6.3 ***Evaluation of linear response***

The dam model was analyzed for the combined effects of static and seismic loads. The static loads consist of the dead weight of the dam, hydrostatic pressure, silt pressure and uplift pressure. The uplift pressure was assumed not to change during the earthquake ground shaking. The result of analysis includes envelopes of maximum stresses, time history of stresses, time history of displacements, and time history of reaction forces at the dam-foundation contact. The envelopes of maximum stresses are used to assess severity and extent of over-stressed regions. The stress time histories are used to compute cumulative duration of stress excursions for comparison with the acceptance limits as per EM 1110-2-6051. Time histories of reaction forces are used to compute instantaneous factor of safety to assess stability conditions of the dam.

6.3.1 *Envelopes of maximum stresses*

Figure 6 (a) to (e) shows envelopes of maximum vertical stresses for the five earthquake acceleration time histories. The results indicate that high tensile stresses generally develop at the heel and toe of the dam.

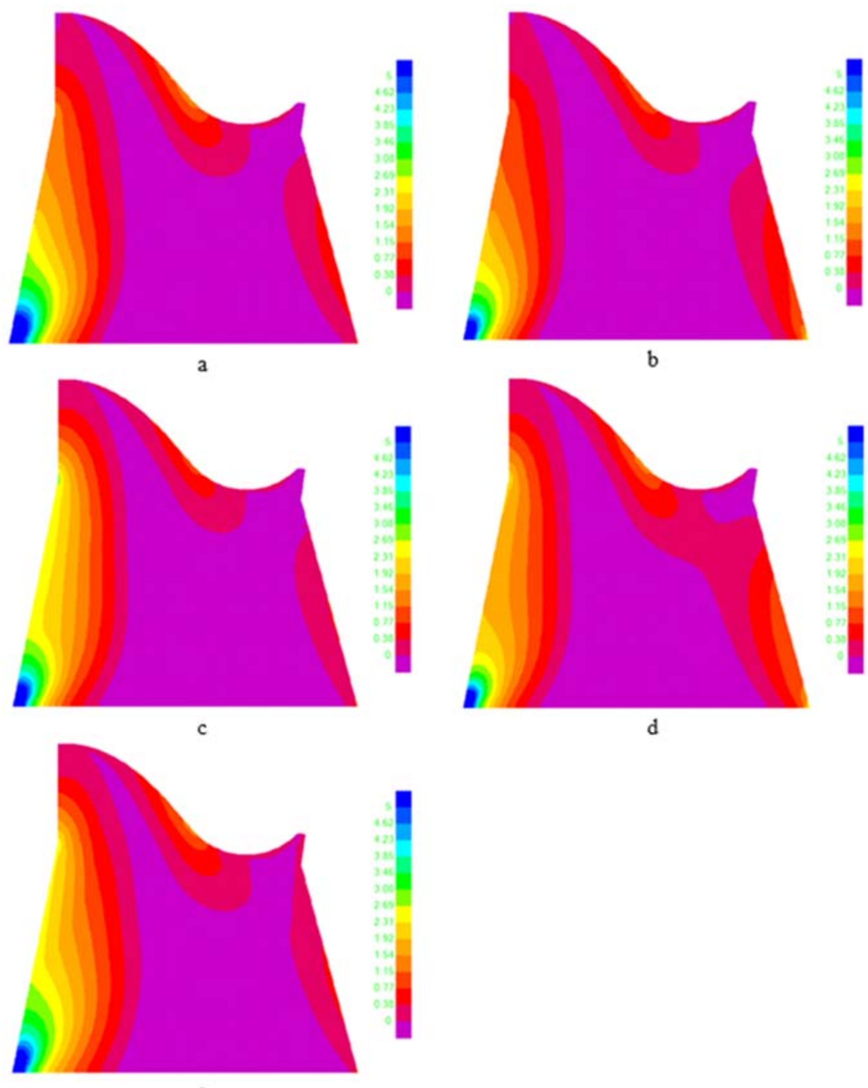

Figure 6 Envelope of vertical stresses from linear time history analysis (Static+TH-Hor+TH-Ver), N/mm^2 (a) TH1, (b) TH2, (c) TH3, (d) TH4, (e) TH5

6.3.2 *Time history of maximum stresses*

Figure 8 shows time history plots of maximum stresses at the heel (Element 1, 3 and 5 at Figure 7) and toe (Element 71) of the dam. For the concrete, a static tensile strength of 2.4 MPa and apparent dynamic tensile strength of 4.8 MPa are taken as the design values. For the computation of cumulative duration at which material strength is exceeded, the stress of 2.4 MPa corresponding to demand-capacity ratio (DCR) = 1 and 4.8 MPa to DCR = 2 are selected. The result show that the DRC at the heel of the dam exceeds 2. This suggests that cracking initiates and propagates from the heel of the dam. Stress generated in element 3 is considerably less than that in element 1, which indicates that the stress is concentrated in a very small area and the result may be a numerical artifact associated with the software's stress distribution method. Analysis result shall be checked with other FEA software to verify this.

6.3.3 *Time history of maximum displacement*

The time histories of horizontal displacements at top of the dam are shown in Figure 6 for the five earthquake records. Table 5 shows maximum horizontal displacement at top of dam for different earthquake records. Largest top displacement of 76 mm is caused by time history 1 while smallest top displacement of 56 mm is given by time history 4.

6.3.4 *Comparison with acceptance criteria*

The acceptance criteria for the linear time history evaluation of gravity dams are defined using the performance curves given in EM_1110-2-6053, in which the percentage of the overstressed regions and cumulative duration of stress cycles above the tensile strength of the concrete need to be below the specified limits. The static tensile strength and the apparent dynamic tensile strength of the concrete represent the range of acceptable tensile stress magnitudes. Knowing the static tensile strength of the concrete, the surface areas with tensile stresses above demand-capacity ratios of 1, 1.2, 1.4, 1.6, 1.8 and 2 were estimated from the stress contour plots. A plot of the result in
Figure 7 shows that the overstressed areas do not exceed the acceptable limit except a little higher for DCR 2, for the five earthquake records. Figure 8 compares cumulative duration of stress cycles for element 1, 3, 5 and 7 at the heel and element 69 and 71 at the toe of the dam with the acceptance curve. For all earthquake records, cumulative duration of element 1 at the heel of the dam exceeds the acceptance limit. This suggests that cracking initiates from the heel of the dam. Cumulative duration of element 3 at the heel of the dam slightly exceeds the acceptance limit while response of element 5 at the heel of the dam is well below the acceptance limit for all earthquake records. Result shows that the elements where the tensile stress exceeds the acceptable limit are concentrated in a very narrow area which might be a result of stress distribution error in part of the FEA software. From the results, it is seen that a non-liner analysis can be performed to further optimize the dam section.

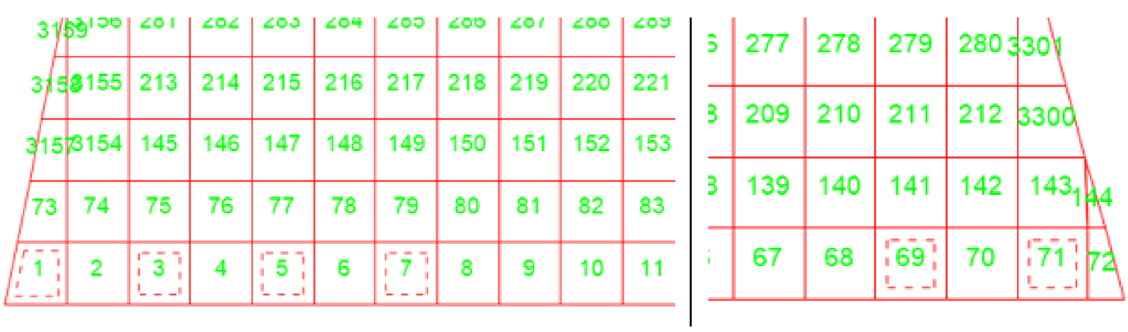

Figure 7 Elements 1, 3, 5 and 7 at heel and 69 and 71 at toe of dam

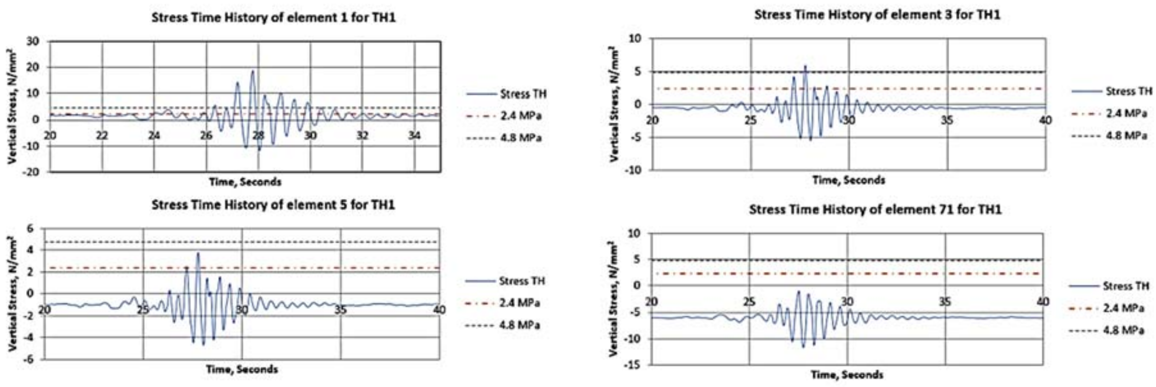

Figure 8 Time history of maximum stresses for element 1, 3, 5 and 71

Table 5 Maximum horizontal displacement at top of dam for five earthquake records

Time History	Maximum Top Horizontal displacement, mm
1	76
2	69
3	73
4	56
5	71

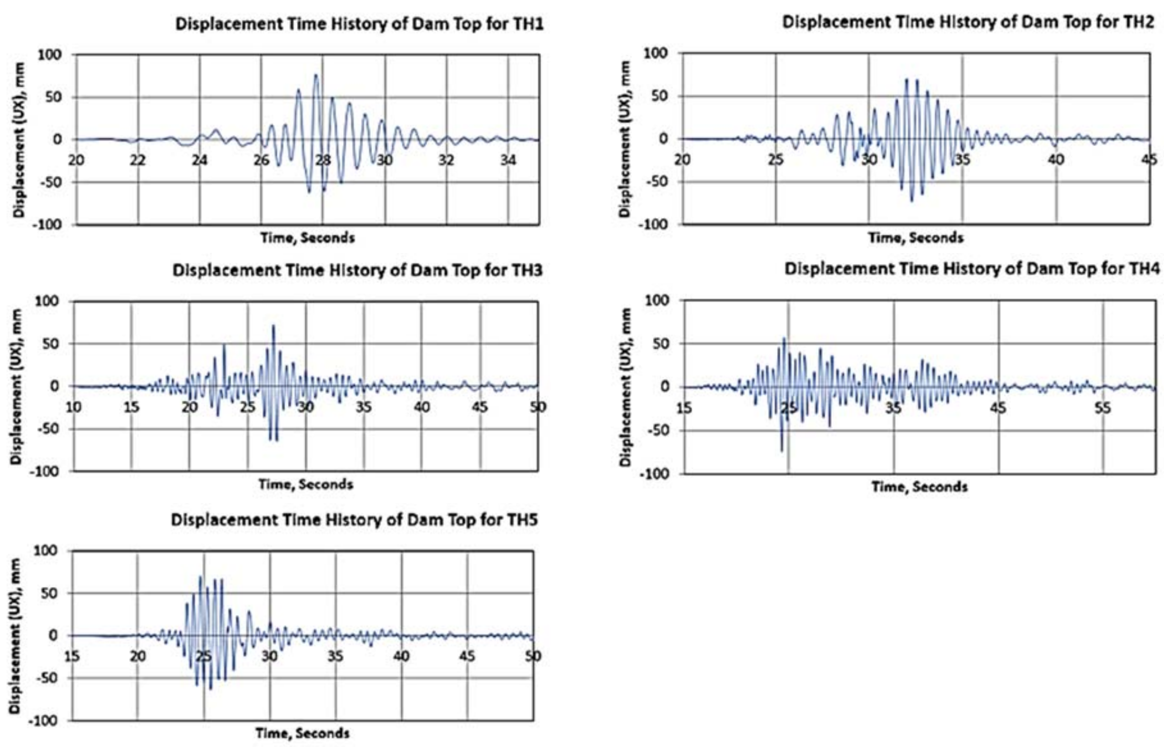

Figure 6 Time history of horizontal displacement at top of the dam

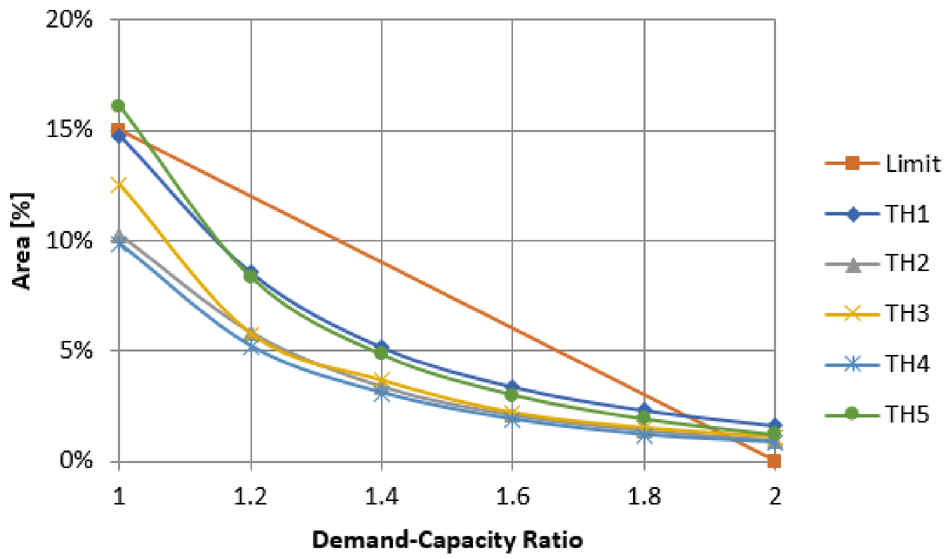

Figure 7 Comparison of percentage of overstressed area with acceptance limit

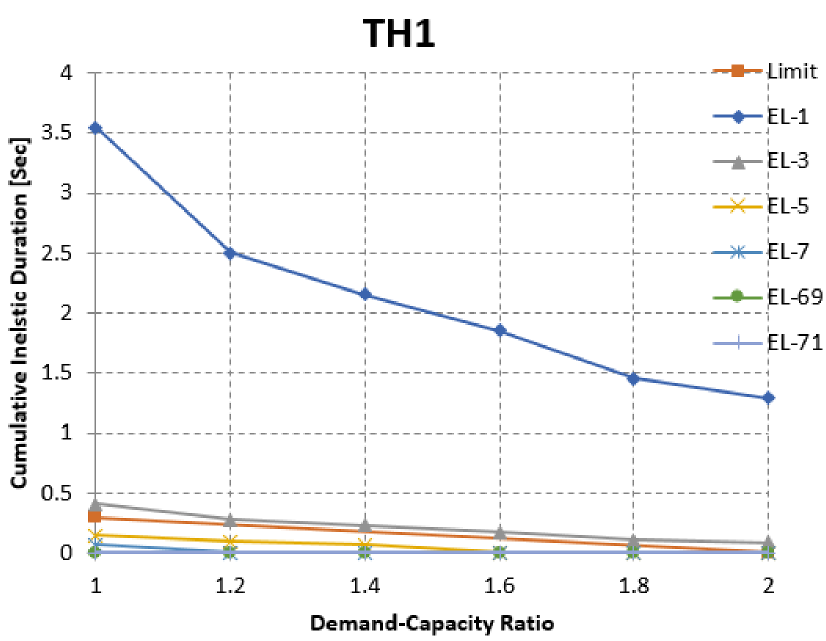

Figure 8 Comparison of cumulative duration of stress cycles with acceptance limits for stresses at the heel and toe of the dam for time history 1

6.3.5 *Seismic stability condition*

The normal and horizontal forces along the dam-foundation contact were obtained to assess sliding stability condition of the dam. Time histories of vertical and horizontal forces were computed by the FEA program for the combined effects of gravity, hydrostatic, uplift and the earthquake loads. Knowing the normal and shear forces and assuming a friction coefficient of 0.9 and cohesion of 800 kN/m^2, the instantaneous sliding factors of safety were computed and shown in Figure 9. The instantaneous factor of safety at the time of zero represents the static sliding factor of safety, which for this example is 2.38. During the earthquake ground shaking the instantaneous factor of safety oscillates above and below the value of the static factor of safety as the magnitude and direction of inertia force changes. Figure 9 shows that the instantaneous factor of safety rarely falls below unity, an indication that the dam will not slide along its base.

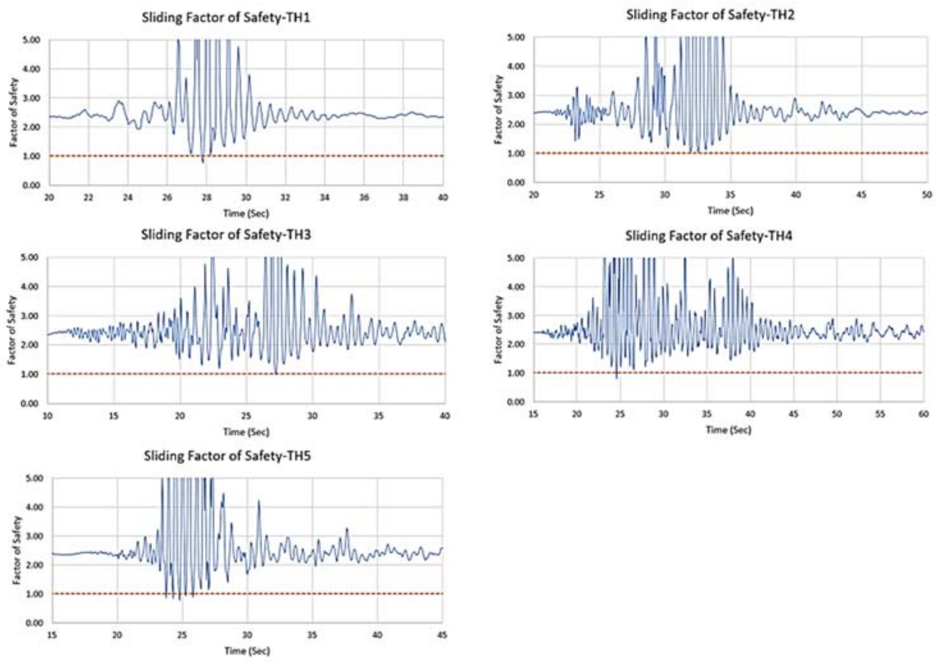

Figure 9 Time history of instantaneous factor of safety for five earthquake records

7 CONCLUSION

Linear time-history analysis was used to assess earthquake performance of an overflow section of gravity dam of Phukot Karnali Hydroelectric Project (480 MW), Nepal. The linear-elastic time-history analysis was employed to gain insight into the dynamic behavior of the dam, to account for transitory nature of earthquake ground shaking. The results of linear-elastic time history analysis were compared with the EM 1110-2-6051 performance acceptance criteria for gravity dams. This comparison indicated that the dam would suffer cracking at limited area of heel and a nonlinear analysis will be necessary for a better assessment of this potential for cracking. The results also showed that instantaneous factor of safety obtained from linear elastic time-history analysis rarely falls below one. The result shows that the dam section can further be optimized by using non-linear time history analysis.

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