Ninth Canadian Conference on
Earthquake Engineering
Ottawa, Ontario, Canada
26-29 June 2007

TWO AND THREE DIMENSIONAL SEISMIC ANALYSIS OF DIRE DAM

B. Yoseph[1] and G.V. Ramana[2]

ABSTRACT

The maximum section of a dam is usually considered in a 2D model (plane strain) to compute the dynamic response of earth dams. The accuracy of the analysis based on plane strain condition is acceptable for dams built in long valleys, whereas for dams built in narrow gorges, a 3D dynamic analysis is required to account for the canyon effect. Seismic excitation of dams in narrow gorges can produce transverse crack in the dam crest, which may pass through core material and create a path for water to flow through the dam core and this can not be captured with a 2D analysis. In this paper, the 3D dynamic response of earth dams is computed and compared with 2D analysis results. Dire earth dam, which is located 40 km northeast of Addis Ababa has been considered as a case study. This dam is a non-homogenous rockfill type with 46 m height and has a crest length of about 700 m. The dynamic material properties were obtained from actual laboratory tests carried out for the construction project. Nonlinear dynamic analysis was performed using FLAC and FLAC3D, the general purpose codes for finite difference analyses. For time history analysis, 2D and 3D models were subjected to synthetic strong ground motions with a *pga* of 0.27g as per the seismic hazard assessment and in conformity with fault length in the vicinity. Some of the key parameters from dynamic response of 2D and 3D models of the dam were compared and the following main conclusions have been drawn from this analysis: (1) Tensile longitudinal stresses on a dam crest are important and they should not be ignored in dam design and 2D modeling can not capture this (2) 2D models may produce conservative results when compared to their 3D counterpart (3) The peak crest accelerations computed for a compatible 2D model of the largest-cross section and from the 3D analysis show that the 3D model under-predicts the values.

Introduction

The structural complexity of earth dams and the high risk associated with (social, economic and environmental) consequences of failure requires very reliable analytical tools to assess their performance, especially when carrying out a safety assessment involving seismic loads. Earth dams subjected earthquakes may suffer the following major damages (Seed 1979):

- settlements and fractures of the dam body;
- freeboard loss up to the limit of overtopping;
- global instability of upstream and downstream slopes;
- reduction of shear strength up to liquefaction of construction and/or of foundation soils;
- differential displacements between embankment, abutments and spillway;

[1]B. Yoseph, Doctorial candidate, Indian Institute of Technology, Civil Engineering Department, Email: yosbirru@yahoo.com
[2]G.V. Ramana, Associate Professor, Indian Institute of Technology, Civil Engineering Department, Email: ramana@civil.iitd.ernet.in

- failure of outlet works crossing the dam body;
- disruption of dam by major fault movement in foundation;
- overtopping of the dam produced by soil masses sliding into reservoir.

Permanent displacements induced by earthquakes can be considered as the integral effect of volumetric and deviatoric plastic strains developed within the dam embankment. Deviatoric strains produce, as an isolated effect, the so called “lateral spreading” of the dam body. Significant crest settlements produce freeboard loss up to overtopping. Even if this limit is not reached, differential settlements may induce other critical types of damage such as fractures within the dam body, which could then jeopardize the water tightness of the structure. Global failure induced by earthquakes might occur if the shells are so steep that even static conditions are close to instability. Seismic forces always produce an increase in instability loads and for some soils a decrease of shear strength as well.

The failure mechanism may involve either the dam body and/or the foundation soils. Other types of damage could be from strength reduction of soils induced by excess pore water pressure. The latter might cause two different phenomena: liquefaction and cyclic mobility. Liquefaction consists of a cyclic shear stress process that induces a reduction in strength not compatible with static stability conditions acting at the end of the cyclic process itself. Cyclic mobility refers instead to a similar process where strength reduction is consistent with static stability. Liquefaction induces high deformation in the earth dam and may cause collapse either during or after the seismic event.

Since in a coupled effective stress method the predictions of the dynamic analysis are strongly affected by the pre-seismic conditions of stress and pore water pressure distribution within the structure, it is necessary to simulate all stages of the dam’s history up until the actual seismic event: construction, first impounding, and service operations.

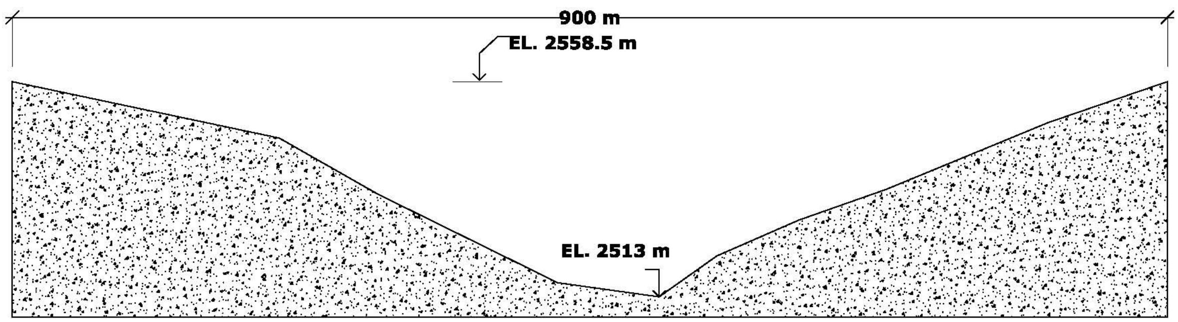
Different researchers have developed methods for seismic analyses of earth dams considering one, two or three dimensional behavior in the order of increasing complexity depending on crest length to height ratio and boundary conditions. Three-dimensional nonlinear response analyses are relatively straightforward extensions of their two-dimensional counterparts. When applied to the back-analysis of earth dams, it realistically represented dam response while accounting for all three components of ground shaking and all modes of vibration (Kramer 1996).

This paper describes the dynamic analysis of Dire dam simulating the actual dam section, geologic and geotechnical conditions, and estimated seismic loading from synthetic ground motion. The maximum cross section of the dam was considered for two-dimensional analysis using Version 5 of *FLAC* (Itasca 2005) and the whole dam was analyzed using the three-dimensional version of *FLAC* (Itasca 2002) to observe the variation of some of the critical response quantities. The Byrne model (Martin et al. 1975, Byrne 1991) was used to simulate the development of excess pore water pressures within the dam and possible occurrence of liquefaction.

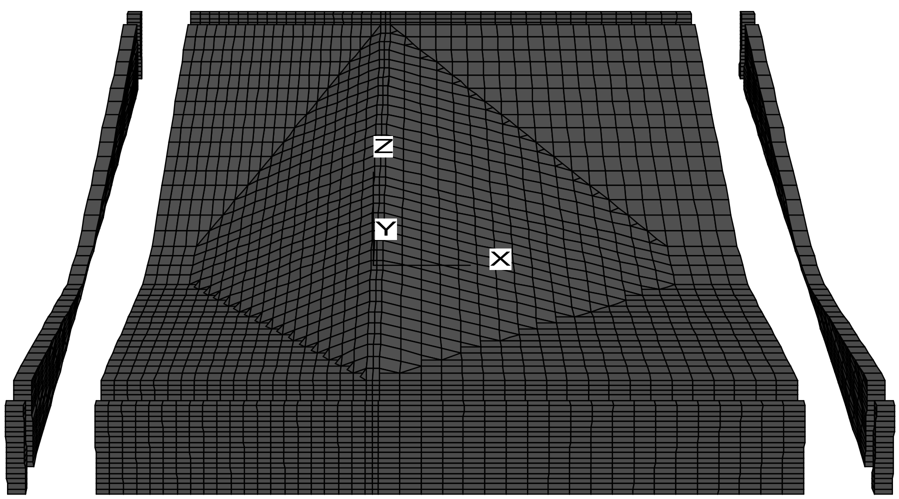
Description of the Dam and Seismic Hazard Estimation

Dire Dam project is located 40 km north east of Addis Ababa on the Addis Ababa – Dessie road. It is envisaged to construct a storage dam facility within the gorge of the perennial Legedadi River. The purpose of this project has been initiated to alleviate the critical water supply shortage for the city of Addis Ababa by supplying additional raw water to the nearby Legedadi water supply treatment plant.

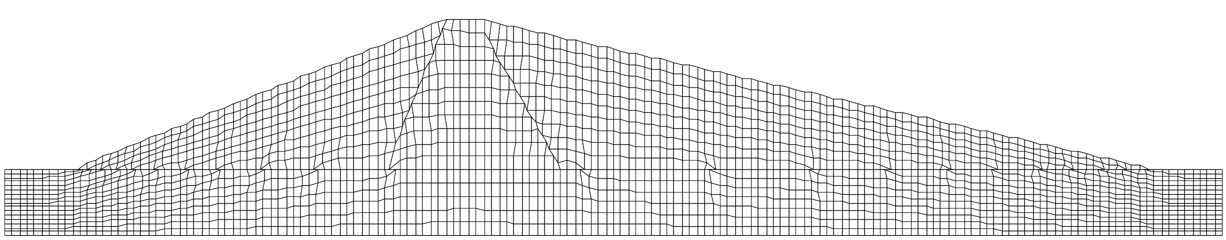
Dire dam foundation and abutment consists of residual clay (decomposition product of tuff), ignimbrite rock and agglomerated basalt beneath the tuff. The coefficient of permeability for residual clay and agglomerated basaltic rock ranges from 10^{-5} to 10^{-8} cm/s. Due to joint plan existence in ignimbrite rock and possible leakage problem, it was removed by excavation. Most of the reservoir valley is covered with dark, highly plastic clay intermixed with colluvial material. The elevated and flat topography of the abutment is characterized by exposure of ignimbrite, which continues within the ridge of the reservoir. Exposure of tuff is also covered substantial portion of the area of the reservoir to left side of the river. The over all height of the dam is taken as 46 m, for which 2.5 m is allocated as the head over the spillway and the remaining 1.5 m for free board. This is a zoned dam consisting of filters and weathered rock material was used as shell. The crest has a width of 7 m and length of 1985 m, of which 1318 m is dyke. The dam is proposed to have curves of radius 410 m which has not been considered for the current analyses. The central impervious core is made of highly plastic clay (CH). The top width of the core is taken to be 3 m and 0.5 m above the maximum water level. The bottom width of the core is 25.50 m. The transition filters provided on downstream of the core is used to avoid migration of fine particles and to reduce the hydraulic gradient. Upstream and down stream slopes are 1:4 and 1:2.5 respectively. The 3D finite difference mesh consists of 20072 elements and the corresponding 2D model is made of 2560 elements. Figs. 1b & c show the meshes used for 3D and 2D models of the dam, respectively.



a) Canyon cross section



b) 3D representation of Dire dam



c) 2D model of maximum cross section of the dam

Figure 1. Canyon shape, 3D and 2D presentation of the dam model.

Dire dam site is located on the west edge of the northern Ethiopian highlands about 22 km east of Addis Ababa. Volcanic material characteristic of the highlands form the bedrock of the region. Tectonic activity during the early development of the East African Rift produced the major valley which extends south west from about the location of Awash river across the country of Ethiopia and into Kenya. The western edge of this rift valley is located approximately 20 km south east of Dire dam site (Dire dam design document, 1994). The region is situated on the marginal zone of 2 to 3 on the seismic risk map of the country (EBCS-8 1995) which categorizes the site as relatively high seismic risk area.

As dire dam site is located upstream of populated area, the maximum credible earthquake from controlling fault in the area is considered for analyses. The closest major fault considered capable of generating strong ground motion at the site is an unnamed fault located approximately 20 km south east of the site according to the project investigation report. This fault is one of the traces comprising the western margin of the East African Rift. The margin of the rift extends several hundred km; however, individual fault traces along the margin are on the order of 30 to 40 km long. The style of faulting expected in the rift is normal-slip due to extensional stresses generally perpendicular to the trend of the rift. Relationships predicting earthquake magnitude based on fault length were reviewed by (Bonilla et al. 1984). Normal fault up to 40 km long is expected to generate earthquake magnitude of 7.0 to 7.2. These predicted maximum magnitudes are in good agreement with the 1961 Kara Kore earthquake in the region.

The geologic conditions of the foundation at the Dire dam site is considered to be soft rock in the classification used by (Campbell and Bozorgna 1994) attenuation relationship of peak horizontal acceleration (*pga*). Using this attenuation relationship the original design has used a mean *pga* for earthquake of magnitude 7 to 7.2 at a distance of 22 km ranging from 0.167g to 0.25g. The acceleration time history developed using specific barrier model (Yoseph and Ramana 2006) for nonlinear analyses of the dam also produced the same order of magnitude of *pga* as shown in Fig. 2. For dynamic analysis, the dam is subjected to synchronous bi-directional ground motion due to high shear wave velocity of the rock foundation material.

Material Properties

The material properties shown on Table 1 were taken from the investigation and design documents of the dam and the correlations were estimated from references for parameters not found as direct test values.

Table 1. Material properties of the dam used for the analyses.

Properties	Foundation	Shell material	Core material
Classification (USG)	Rock	GW - GC	CH
Drained cohesion, C' (kPa)	-	20	90
Saturated cohesion, C_{sat} (kPa)	-	0	45
Drained friction, ϕ'	37	30	20
Dry density, ρ_d (kg/m^3)	-	1940	1370
Saturated density, ρ_{sat} (kg/m^3)	2760	2200	1730
Permeability, k (m/sec)	1.143E-08	8.87E-07	9.93E-08
Porosity, n	0.30	0.371	0.30
Shear wave velocity, Vs (m/s)	1765	321	-
Shear modulus, G_{max} (kPa)	8.6E06	1.96E05	1.0E05
Bulk modulus, K(kPa)	1.86E07	4.23E05	2.0E05

Due to absence of actual test of damping for the dam materials, 5% rayleigh damping was adopted at characteristic frequency of 3 and 2.1 Hz for 3D and 2D models respectively. The methods followed to obtain these characteristic frequencies are detailed in the next section.

Dynamic Analysis

The following procedures have been followed to for Dynamic Mechanical Simulations of the dam for seismic response as per the recommendation of FLAC manual (Itasca 2005) and other references:

1. It was ensured that model conditions satisfy the requirements for accurate wave transmission.
2. The canyon is assumed to be rigid. The alluvial foundation has been modeled as part of the dam structure.
3. Appropriate mechanical damping was specified to be representative of the materials and input frequency range.
4. Dynamic loading and boundary conditions were applied. The acceleration time history was filtered to satisfy the requirements for accurate wave transmission.
5. Hydrodynamic load was neglected as it is not critical for earth dam structures (Chopra, 1967). Hydrostatic pressure of the water of the upper pool was applied to the upstream face of a water tight element as a surface load.
6. Displacement, pore water pressure generation and acceleration history were monitored during the dynamic analysis of the model.

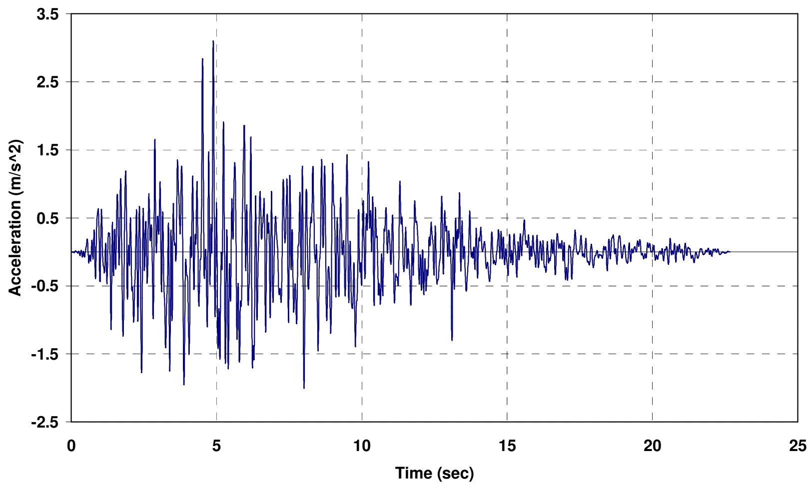


Figure 2. Input acceleration time history developed as rock out crop motion.

For many problems, the important frequencies are related to the natural mode of oscillation of the system. Examples of this type of problem include seismic analysis of surface structures, such as dams or dynamic analysis of underground excavations. The longest wavelength (or characteristic length or fundamental wavelength) depends on boundary conditions. If a wavelength for the fundamental mode of a particular system cannot be estimated in a simple way, then a preliminary run may be made with zero damping (FLAC manual Itasca 2005). A representative natural period may be estimated from time histories of velocity or displacement. Due to smaller computation time requirement, this method was used to estimate the fundamental frequency of the dam in 2D model which resulted 2.1 Hz.

The characteristic frequency of the dam for 3D analysis was estimated using the following relationship (Kramer 1996):

$$f_1 = \frac{1}{2\pi}(2.404Vs/H) \qquad f_2 = \frac{1}{2\pi}(5.52Vs/H) \qquad f_3 = \frac{1}{2\pi}(8.6544Vs/H) \tag{1}$$

f_i is the frequency (Hz), V_s is shear wave velocity (321 m/s^2), and H is height of the dam (46 m).
From the above, the first three natural frequencies of embankment are estimated as 3, 6 and 10 Hz. Hence, f_{max} (cutoff) is 10 Hz which demands maximum mesh size of 3.21 m. Since the bulk of the energy

in a seismic event is carried in the first three natural frequencies, a cutoff frequency of 15 Hz is used in the analyses.

Considering the above inputs and assumptions, seismic analyses of Dire dam at normal impounded level was conducted using 2D and 3D representation and applying nonlinear effective stress method incorporated in FLAC software. Some of the key outputs assuming that the dam material characterized with Mohr coulomb's behavior are listed below. It was also analyzed using Finn model assuming that the material may under go liquefaction due to seismic loading, but all the indicative parameters revealed that no such situation prevail under the anticipated seismic hazard level and therefore no further reference is made in this paper to liquefaction induced damage.

Results and Discussion

The assumption of plane-strain conditions (which forms the basis of the 2D analysis that is most widely used in practice) is valid only for infinitely long dams subjected to a 'synchronous' base excitation (i.e., identical motion of all points along the base). For dams built in narrow valleys, as is often the case with rockfill dams, the presence of relatively rigid abutments creates a 3D stiffening effect, whereby natural periods decrease and near crest accelerations increase sharply as the canyon becomes narrower (Gazetas and Dakoulas 1991).

This paper presented analyses of rockfill dam using 2D and 3D versions of nonlinear effective stress analyses using FLAC software. Using relatively simple models, rayleigh damping and the Mohr model, most of the significant phenomena anticipated at the site during a seismic event were tried to be represented realistically. These included the use of synthetic ground motions, the transmission of the resulting shear waves through the various media and the associated amplification due to geometry and material characteristics, the development of deformation, and the generation of excess pore water pressures.

Fig. 3 shows the comparison of the time histories of acceleration at the crest of the dam. The two models resulted in closer pga of 5.5 and 5.15 m/s^2 for 2D and 3D cases respectively. But as to the frequency content and the temporal location of peak is concerned, the results are entirely different. The 3D output filtered high frequency components more and also revealed the peaks uniformly for some definite durations. Whereas, in the 2D case spiky appearance of the peaks may tempt to imply that it is more unlikely in reality. This was also observed by Gazetas and Dakoulas, for the Kisenyama dam in Japan, a 95 m clayey-core rockfill dam built in 1969. On the other hand, in a very narrow canyon, the peak crest acceleration can be more than two times the value predicted for an infinite-long dam, i.e. under 2D plane-strain conditions.

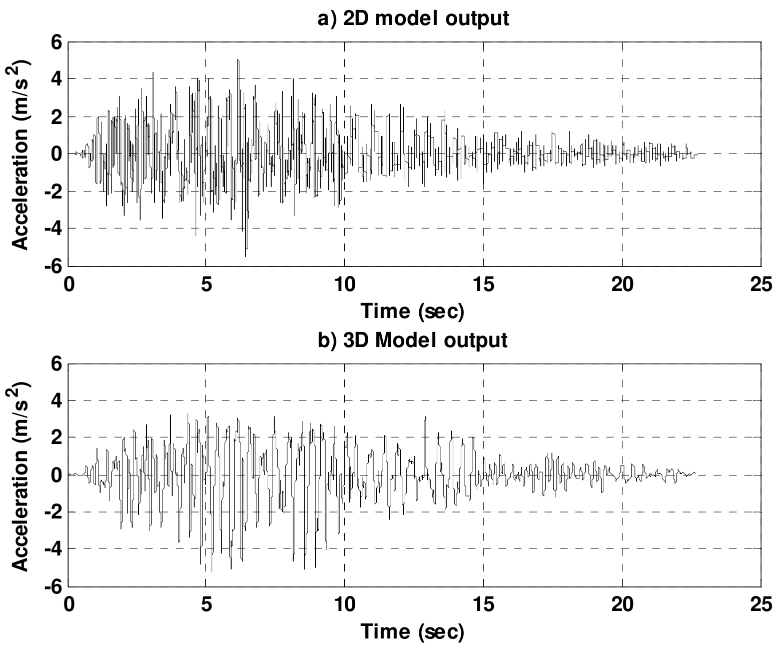


Figure 3. Acceleration time history at crest of the dam monitored at similar locations.

Previous investigation comparing 2D and 3D analyses (Majia and Seed 1983) showed that dams in steep canyons are more sensitive to boundary conditions specifically for acceleration responses and concluded that plain strain analyses of the maximum section apparently could not simulate correctly the behavior of the embankment under seismic loading. In the case of Dire dam, the canyons are so gentle and accordingly the acceleration responses at the crest predicted from the two model do not show significant deviation.

In Fig. 4, downstream displacements at the crest were shown with maximum permanent deformation of about 45 and 20 cm for the 2D and 3D cases, respectively. The 2D analyses predicted higher values possibly due to reduced stiffness resulting from absence of three-dimensional confinement.

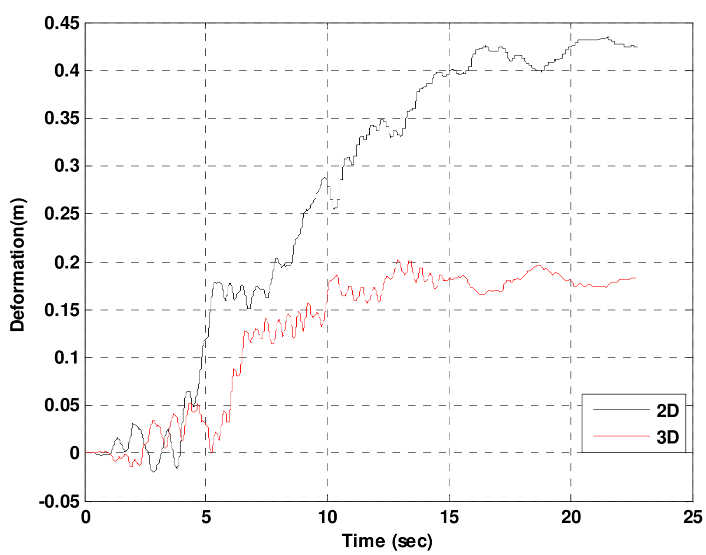


Figure 4. Transverse displacement time history of the dam at the crest.

The vertical displacement histories recorded at similar locations in the respective models is shown in Fig. 5 (about 3.5 cm for 2D and 1.5 cm for 3D case). But in the contour plots of vertical displacements, at some distance from the crest the 2D counterpart generated a 15 cm deformation which again is on the higher side.

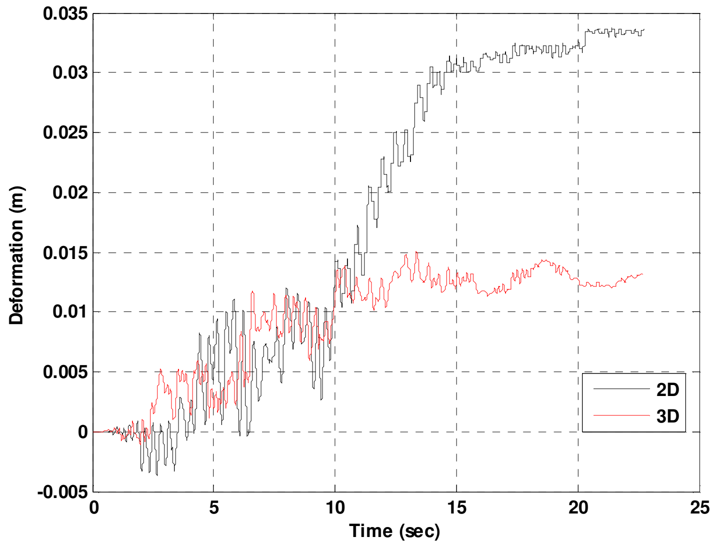


Figure 5. Vertical displacement time history of the dam at the crest.

Regarding transverse velocity in the same history location shown in Fig. 6, both models produced similar result of 0.35 and 0.36 m/s as peak values. The velocity history also shows that it is converging to zero at the end of shaking which along with displacement history implies that no liquefaction has occurred.

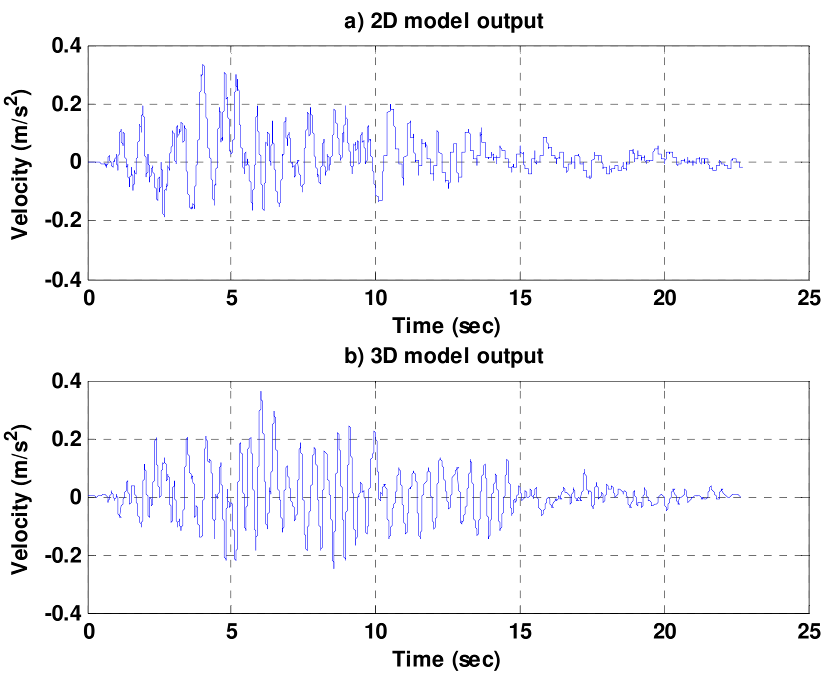


Figure 6. Transverse velocity time history monitored at the crest of the dam.

In Fig. 7 the deformation along the dam axis has been shown for the 3D model and revealed that extension of about 1.5 cm occurred near the crest of the dam. This is the important ingredient of the 3D analyses to check the safety of the dam for possible crack development and the consequent leakage and “piping” problems due to seismic loading.

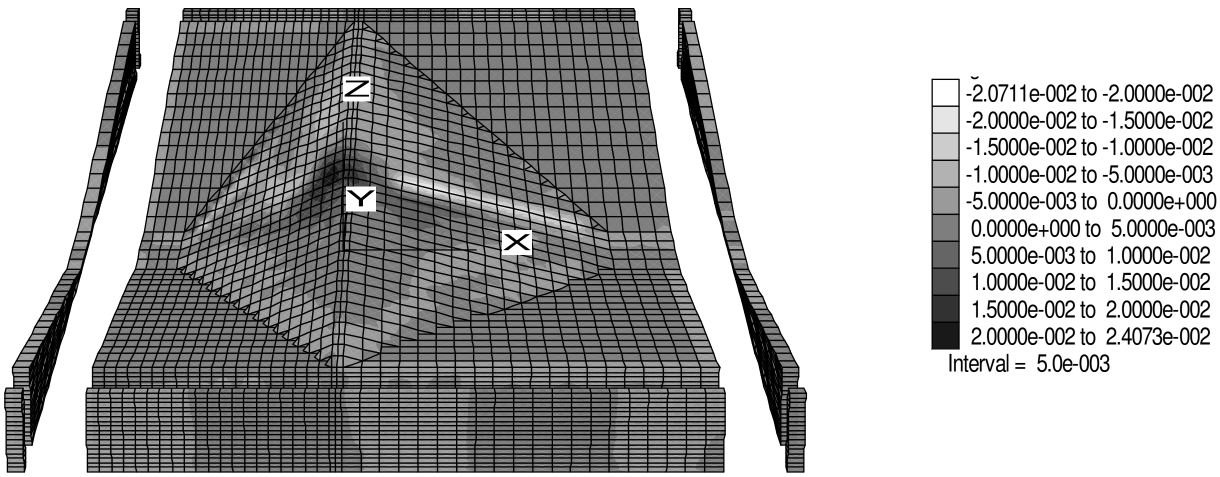


Figure 7. Displacement contours along the dam axis for the 3D analysis.

Conclusions

The dynamic response and shear stresses for two and three-dimensional models were compared and the following main conclusions have been drawn from this research:

1. Tensile longitude stresses on dam crest are important and they should not be ignored in dam design.
2. 2D models usually produce conservative results along the plane of the model compared to their 3D counterpart.
3. The peak crest accelerations, velocities and displacements computed for a compatible 2D model of the largest-cross section and from the 3D analyses were compared and the latter predicted lower values.
4. The seismically induced displacement of the dam during the design as well as maximum credible earthquakes were estimated to be less than 0.5 meters and hence the dam may be considered safe against overtopping for the available freeboard.

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