SEISMIC RESPONSE AND DYNAMIC DEFORMATION ANALYSIS OF SHUR RIVER DAM

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ABSTRACT

Seismic load has induced extensive deformations or caused failure in many dams during past earthquakes. As such, it is considered to be the governing load for the design of dams in the regions of moderate to high seismicity by dam designers. This paper discusses the methods and results of the seismic deformation analyses of an 84 m high water retaining rockfill dam with an asphaltic concrete core, located in a region of high seismicity in Iran. The purpose of the dam is to store raw and processed water for use in a copper production complex. The maximum crest settlement of the dam under three earthquake loadings was evaluated using simplified procedures and numerical dynamic analysis. Plane strain dynamic deformation analysis of the dam has been performed using the commercial software FLAC and its built-in Mohr-Coulomb elastic-plastic model. The main objective of these analyses has been to identify the earthquake induced movements that could lead to uncontrolled release of water. The predicted deformations are compared with the results of simplified analyses and the observed displacements in other embankment dams during past earthquakes. The relative movement of the asphaltic core with respect to the rockfill material on either side of the core was also studied.

1 INTRODUCTION

Asphaltic core rockfill dams (ACRD) have gained significant attention in the past decade as dam engineers have become increasingly interested in the design of this type of dam. The main reasons for this have been the deformation characteristics, self-healing nature and constructability of asphalt cores in most climatic conditions, as well as the satisfactory performance of rockfill material under all types of loading conditions.

Rockfill dams have been shown to be able to withstand strong earthquake loading (Foster *et al,* 1998 and ICOLD, 2001). No serious incident has been reported due to a severe earthquake although some permanent crest settlements and slope deformations have occurred due to earthquake excitation (Swaisgood, 2003). This makes rockfill embankments ideal for dams in earthquake prone regions. Embankment dams generally exhibit some crest settlements due to accumulation of settlements of underlying layers during construction. In the case of earthquake loading, however, both horizontal and vertical deformations may be expected. The impervious core of a rockfill dam must have sufficient flexibility to adjust to the potential foundation settlements and/or differential settlements, which are normally accompanied by large tensile and shear stresses, without cracking or noticeable increase in permeability. An asphaltic concrete core is therefore considered a suitable option in regions with high seismic activity. The bitumen content of the asphalt core can be adjusted so that the core remains visco-elastic-plastic within the range of changes in local temperature. A visco-elastic-plastic material such as asphalt has a self-healing ability when subjected to differential settlements and therefore remains impervious. Utilisation of asphalt in a core, as compared with a clay core, has the potential to reduce seepage and increase the factor of safety against instability.

Asphaltic concrete has also been used as a membrane on the upstream face of embankment dams. However, the tensile stresses and strains along an upstream face membrane, generated due to deformation of the rockfill, are much greater than those generated in a central core (Ghanooni and Mahin-Roosta, 2002). The earthquake induced deformations of embankment slopes have little effect on the central core but would cause large deflections and tensile strains in the upstream face. An upstream face membrane is also exposed to weathering due to temperature variations, oxidation and radiation, which may result in degradation, brittleness and cracking of asphaltic concrete. Therefore, it is easier to satisfy the design requirements for the asphaltic concrete to be used as a core rather than as an upstream face membrane.

Different methods of pseudo-dynamic and dynamic analysis have been proposed and used for deformation analysis of embankment dams under earthquake loading. Among these are the original contribution of Newmark (1965), which was later modified by Makdisi and Seed (1978). In these methods sliding of a soil mass along a potential failure surface was simplified into sliding of a rigid block due to horizontal earthquake acceleration. The acceleration was estimated at different levels using an equivalent linear earthquake response analysis. When the inertia force of the earthquake exceeds the yield resistance of a block, movements begin, and when the inertia force changes its direction, movements stop. Improvements in numerical methods in recent years have resulted in application of non-linear elasto-plastic models to investigate the dynamic response of embankment dams to seismic loads.

The response of asphaltic concrete cores in rockfill dams due to earthquake loading has not been well understood, although these dams have shown excellent performance during construction and operation (Creegan and Monismith, 1996). Deformation analyses of asphaltic core dams under earthquake loading were carried out by Valstad *et al*. (1991), and Meintjes and Jones (1999) using a Newmark-based method. Ghanooni and Mahin-Roosta (2002) used a non-linear elasto-plastic model in dynamic analysis of a typical 115 m high dam. They observed a large relative settlement between the asphalt core and the transition zones on either sides of the core. The behaviour of the asphalt core remained elastic while the transition zones exhibited large deformations and behaved as a plastic material. The maximum shear strain in the core was an order of magnitude smaller than that of transition zones around the core. A Newmark-based analysis showed larger displacements and widespread cracking of the core, which was in complete disagreement with the results of the non-linear numerical analysis. Feizi-Khankandi *et al.* (2009) also performed a dynamic deformation analysis of an asphaltic core rockfill dam and found a significant relative vertical displacement between the core and the embankment. A model test on a shaking table confirmed this behaviour. The residual shear strains after earthquake were estimated to be very small in the asphaltic core (0.5%) as compared to those of the transition zones around the core. It was found that the Newmark method gives smaller permanent displacement than that obtained by non linear deformation analysis. Baziar *et al.* (2009) conducted dynamic deformation analyses and centrifuge tests on a prototype ACRD subjected to an impact load (drop weight) at the base of the embankment. The results of the deformation analyses corresponded well with the observations from the centrifuge tests, and the asphalt core showed the same behaviour in both numerical and centrifuge models.

The dynamic response of an asphaltic core rockfill dam under earthquake loadings is presented in this paper. The geometry, material composition of the dam and their properties are given in the next section followed by a description and the characteristics of the three earthquake loadings selected for the seismic deformation analysis of the dam. The methods used for the deformation analyses of the dam are briefly explained. Finally the results of the analyses are presented in terms of magnitudes and patterns of deformations and conclusions are drawn.

2 DESCRIPTION OF THE PROBLEM

The Shur River Dam is an asphaltic concrete core rockfill dam which is currently under construction in a high seismicity area in south east of Iran. The maximum height of the dam at crest, RL 2364 m, is approximately 84 m from the river base, with the full supply level at 2360 m. The crest width is 10 m with a 0.6 m camber. The dam is founded on volcanic bedrock formed by inter-bedded zones of andesite and tuff. The upstream and downstream slopes are 1.75:1 (H:V) and 1.5:1 (H:V), respectively. The embankment consists of a 0.6 m wide central asphalt core, transition zones on either side of the core, free draining rockfill and coarse rockfill, as shown in Figure 1. A downstream rock toe with a slope of 4:1 (H:V) has also been incorporated for flood protection.

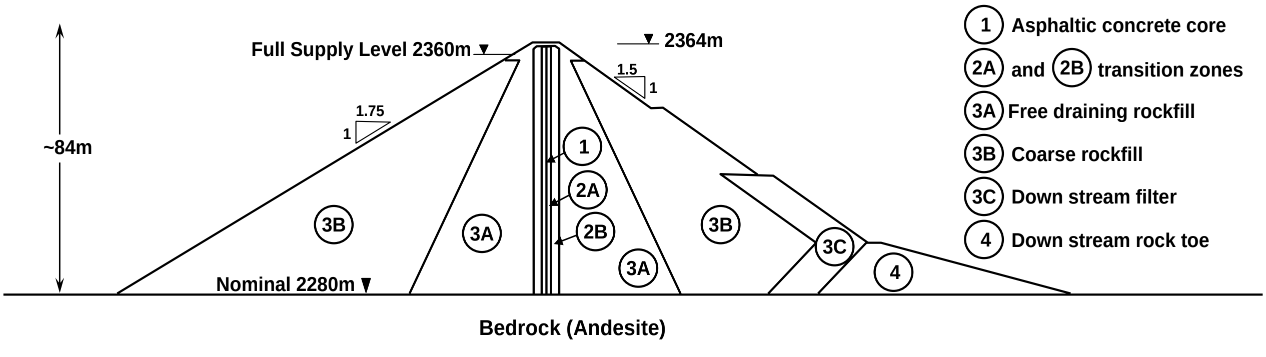

Figure 1: Typical Section of the Dam.

3 SELECTION OF DESIGN EARTHQUAKES

The dam is located in a region of high seismicity and consequently the seismic action was considered to be the governing load case. Two levels of earthquake loading were considered in the analysis and design of the dam, as recommended by ANCOLD (1998):

- **Operating Basis Earthquake (OBE)**: It represents the level of ground motion at the dam site at which only minor damage is acceptable. Such damage should be superficial rather than structural.

The dam, appurtenant structures and equipment should remain functional and damage easily repairable from the occurrence of earthquake shaking not exceeding the OBE. An annual exceedance probability (AEP) of 1 in 500 was adopted for the OBE based on ANCOLD guidelines (1998) and a 10 percent chance of exceedance in 50 years.

- **Maximum Design Earthquake (MDE):** It will produce the maximum level of ground motion for which the dam should be designed or analysed. It is a minimum requirement that the impounding capacity of the dam be maintained when subjected to that seismic load. Due to the extreme importance of the Sarcheshmeh mine to the economy of the region and the risk imposed to the life of people in the downstream villages, the Maximum Credible Earthquake (MCE), an event having PGA=0.8 g and associated magnitude of M=7.5, was taken as the Maximum Design Earthquake (MDE). The MCE corresponds to an event with a return period of 10,000 years. For the MDE event, a factor of safety less than 1.0 (using pseudo-static screening) is acceptable provided that a comprehensive analysis can demonstrate breach would not occur.

Three earthquakes were recommended by the site seismology report and were adopted in the seismic deformation analysis. They are those of the Loma Prieta earthquake, which occurred in 1989 in California, the Cape Mendocino earthquake, which occurred in 1992 in California, and the Nahanni earthquake which occurred in 1985 in Canada.

Pseudo-static analyses of the dam under MDE earthquake loading indicated that instability of both upstream and downstream slopes would theoretically be expected. On this basis seismic response and deformation analyses of the maximum height section were carried out in order to predict the magnitude and pattern of any resultant deformation.

In the deformation analyses only horizontal components of the acceleration-time histories were considered, of which the most severe component was used for the modelling. For each earthquake, a Fourier amplitude spectrum was derived to identify the dominant range of frequencies. The acceleration time history and Fourier amplitude spectrum derived for the three earthquakes are presented in Figure 2.

4 MATERIAL PROPERTIES

4.1 OVERVIEW

The properties of materials used in different zones of the dam were obtained from site investigations, laboratory tests, published literature, established correlations and previous experience. A summary of the properties used in the analyses are presented in Table 1. The discussion following the table provides background information regarding the selection of some of these parameters.

Table 1: Summary of Material Properties.

Material	*Density (kg/m^3)*	*c′ (kPa)*	*ϕ′ (°)*	*υ*	*G_{max} (MPa)*	*ξ %*	*Modulus Reduction & Damping Curve*
Asphalt	2450	2000	0	0.49	40	1	-
Zone 2	2050	0	38	0.3	$19.8\ \sigma'^{0.5}_m$	1	Shake2000 (2003)
Zone 3, 4	2300	τ - σ Function		0.23	$37.4\ \sigma'^{0.5}_m$	1	Shake2000 (2003)
Bedrock	2450	1	60	0.23	2,700	1	Shake2000 (2003)
c′ is cohesion, ϕ′ is friction angle, υ is Poisson's ratio, G_{max} is small strain shear modulus, ξ is initial damping ratio, τ is shear stress, σ is normal stress and σ'_m is mean effective stress.							

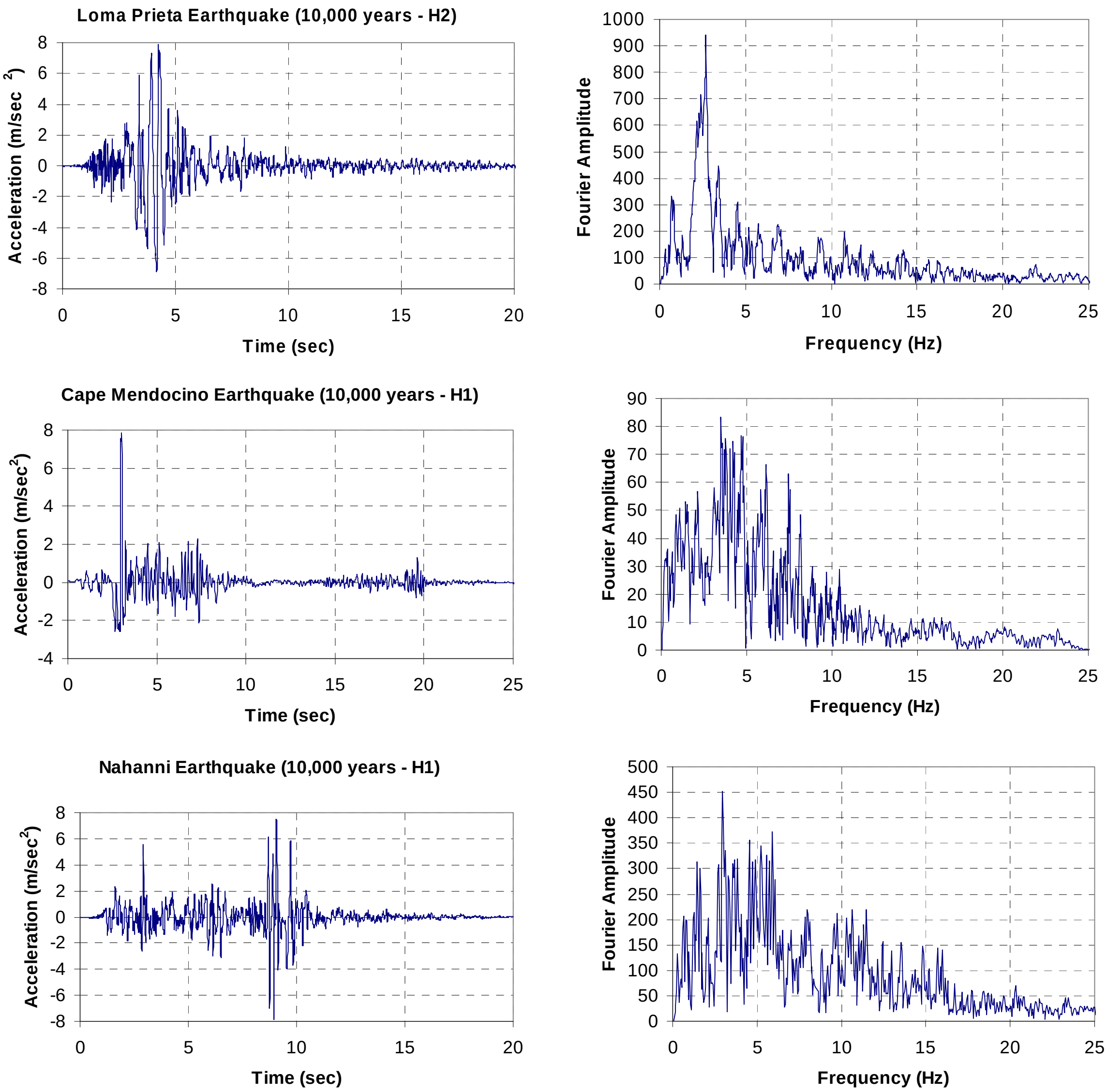

Figure 2: Acceleration Time Histories and Fourier Amplitude Spectra of the Design Earthquakes.

4.2 ASPHALT CORE

The asphalt mixes used as a central core are similar to continuously graded road mixes, but the desirable characteristics are changed from mixes with high strength and high stiffness to those with low permeability and high flexibility. These characteristics can be achieved using higher bitumen contents and higher filler contents compared to those used in road mixes. Permeability is the critical criterion for an asphalt core, and is often evaluated indirectly using the air void content of the mix (Hoeg, 1993).

Cyclic tests on samples of asphalt core showed that the asphalt samples deform in a linear elastic manner under cyclic shear stresses imposed by an earthquake loading, even if the samples are subjected to large initial shear stresses (Breth and Schawab, 1973). Similar conclusions were made by Wang (2004) who performed an extensive range of laboratory testing on samples of asphaltic concrete. During these tests, the cyclic strain amplitudes remained virtually constant even for a large number of cycles, i.e., no material degradation occurred. However, samples exhibited residual strains due to creep under the average static stress. Feizi-Khankandi *et al.* (2008) conducted monotonic and cyclic triaxial tests on samples of asphalt concrete, where no significant degradation on the specimen behaviour was detected even after 10000 cycles. They also concluded that the damping ratio increases with increasing dynamic strain at lower stress ratio, but remains constant at higher stress ratio. Temperature was identified to be the main factor affecting degradation of shear modulus.

For the analysis a cohesion of 2 MPa was adopted from the results of triaxial tests conducted by Höeg (1993), and a conservative friction angle of $\phi = 0^o$ was adopted. A low shear modulus of 40 MPa was assumed for the asphalt core based on the triaxial test results conducted by Jones *et al.* (1993) on a mix with a similar bitumen content. The Poisson's ratio was assumed to be 0.49 to model the constant volume elastic deformation characteristic of the Asphalt core.

Recent triaxial testing of production asphalt samples from the Shur River Dam (by Norwegian Geotechnical Institute) has confirmed the adopted asphalt properties and also indicated that the asphalt remained ductile up to a large strain (around 4%) and the volume dilation (expansion) was negligible.

4.3 ROCKFILL

The rockfill used for construction of the embankment and filters are mainly sourced from an andesite basalt quarry. The rockfill material has a friction angle of $\phi' = 48^o$ at low confining stresses, which reduces to 40^o at higher confining stresses at the base of the embankment. This negates the dilation effect and therefore a dilation angle of zero was adopted for the analysis.

The small strain shear modulus, G_{max}, for the rockfill was taken as a function of mean effective stress, σ'_m , using the empirical relationship reported in Kramer (1996):

$$G_{max} = 220K_{2max} (\sigma'_m)^{0.5} \quad \text{(kPa)} \qquad \textbf{(1)}$$

K_{2max} depends on the quality and relative density of the material. For a gravel, K_{2max} ranges from 80 to 180. Zone 3 rockfill is considered to be of good quality and well compacted, consequently a K_{2max} value of 170 was adopted. A Poisson's ratio of $\nu = 0.23$ was adopted for this zone.

4.4 TRANSITION ZONES/ FILTERS

For Zone 2 the small strain shear modulus was derived from Equation (1) using a K_{2max} value of 90. A typical Poisson's ratio of 0.3 was adopted for this zone.

4.5 BEDROCK

The small strain shear modulus and Poisson's ratio of the igneous bedrock were determined using the S (shear) and P (compressional) wave velocities measured in seismic refraction tests over the foundation area.

5 STATIC STABILITY ANALYSIS

Three different loading conditions were considered for static stability analysis of the dam; full supply level, peak flood level and minimum operating level. The factors of safety obtained from the static stability analyses are presented in Table 2. The analyses resulted in a minimum factor of safety of 2.02 and 2.10 for the downstream and upstream slopes respectively, indicating the level of conservatism of the geometry under different static loading conditions.

Table 2: Factors of Safety Obtained from Static Stability Analyses

Loading conditions	Upstream Slope	Downstream Slope
Full Supply Level	2.38-2.42	2.05-2.08
Peak Flood Level	2.44-2.49	2.02-2.04
Minimum Operating Level	2.10	Not analysed

6 SEISMIC DEFORMATION ANALYSIS

6.1 GENERAL

A pseudo static seismic analysis is not believed to provide an accurate assessment of the behaviour of embankment dams. However, this method was used as a screening tool to determine the necessity and extent of further detailed analyses. The critical factor of safety resulting from the MDE loading was less than one, indicating that some deformation could be expected. The analysis also identified that the upstream slope is likely to be the critical slope under the MDE seismic loading. Therefore, detailed deformation analysis was concentrated on the upstream embankment slope under the MDE loading.

Detailed seismic deformation analyses of the embankment were performed using the simplified method proposed by Newmark (1965), the simplified method proposed by Makdisi and Seed (1978), and a numerical dynamic analysis using the software package FLAC (Itasca, 2005).

6.2 NEWMARK APPROACH

Newmark (1965) proposed a method for evaluation of permanent crest settlement of embankments subjected to earthquake loading. This method is based on sliding of soil mass along an inclined failure surface due to the inertia forces. Sliding would be initiated when the inertia forces exceed the resistance of the shearing sliding mass and would stop once the inertia forces are reversed. A series of seismic deformation analyses of the embankment was conducted using the Newmark (1965) approach. Quake/W and Slope/W programs (Geo-Slope International, 2005) were used to evaluate the deformation of the embankment during the earthquakes. Quake/W was utilised to calculate the internal stresses in the embankment at predetermined time steps during the earthquakes. Slope/W was utilised to estimate the factor of safety of the embankment along a critical surface based on the internal stresses generated by Quake/W. Slope/W then applies the Newmark (1965) approach to estimate the permanent deformation along a critical failure surface as a function of time.

The total deformation for the most critical failure surface, which passes through the upstream toe, is evaluated to be 0.17m. However, it is worth noting that the calculated deformation is sensitive to the value of the shear modulus of the rockfill. Sensitivity analyses of the embankment with different values of the shear modulus indicated an upper bound value of 0.38m for the permanent deformation of the dam.

6.3 MAKDISI AND SEED APPROACH

Makdisi and Seed (1978) extended the Newmark method by including the dynamic response of the embankment in the analysis process. This method was undertaken to determine the permanent deformation of the embankment under different earthquake loadings. Initially the acceleration record of the embankment crest was obtained by Quake/W together with peak acceleration at various depths below the crest. The yield accelerations of predominant failure surfaces were obtained using Slope/W. The estimate of deformation for each failure surface was then obtained by the relationship given by Makdisi and Seed (1978) between estimated deformation, yield acceleration, the corresponding maximum acceleration at a depth and the earthquake magnitude.

The maximum deformation of the embankment was estimated to be 0.49m and would occur in the upper quadrant of the embankment upstream slope. This value is larger than the upper bound crest settlement obtained by the Newmark method.

6.4 ELASTO-PLASTIC DYNAMIC ANALYSIS

More accurate prediction of deformation patterns and crest settlements can be obtained using an elasto-plastic dynamic analysis. A two-dimensional plane strain dynamic deformation analysis of the embankment was performed using the software FLAC 2D (Itasca, 2005). The rockfill, transition zones and the asphalt core were assumed to obey the Mohr-Coulomb failure criterion while the rock bed was assumed to remain elastic. The model required stiffness and strength parameters for all materials. The variation in stiffness of the rockfill and transition zones at different depths was considered in the modelling based on the relationship given in Equation (1). A non-associated flow rule was assumed for the rockfill and transition zones, assuming the dilation angle of the material was equal to zero.

The finite difference model prepared for the analysis of the dam is shown in Figure 3. The embankment model was generated in three stages to better simulate the construction stages. The initial state of stress within the embankment and its foundation was established, as well as the state of pore water pressure within the embankment. The water level was considered to be at the full supply level, RL 2360m. The distribution of pore pressure within the embankment and in the bedrock was evaluated by the software.

The earthquake motions were applied to the model base as shear stress boundary conditions in order to establish “quiet” boundaries, using a method described in the FLAC manual (Itasca, 2005). To limit the minimum size of the elements in the model the earthquake records had to be filtered for the high frequency content. Numerical distortion of the propagation of wave could occur in a dynamic analysis if the spatial element sizes are coarse as compared to the propagating wave length. Kuhlemeyer and Lysmer (1973) showed that an accurate prediction of displacement could be achieved if more than 4 elements per wave length are allowed. It was noted that most of the energy in the input motions was at frequencies less than 10, 15 and 20 Hz for Loma Prieta, Cape Mendocino and Nahanni earthquakes respectively. Therefore the seismic records were filtered to remove frequencies greater than these values; thereby a more reliable prediction could be achieved using a practically possible minimum size for the finite difference elements.

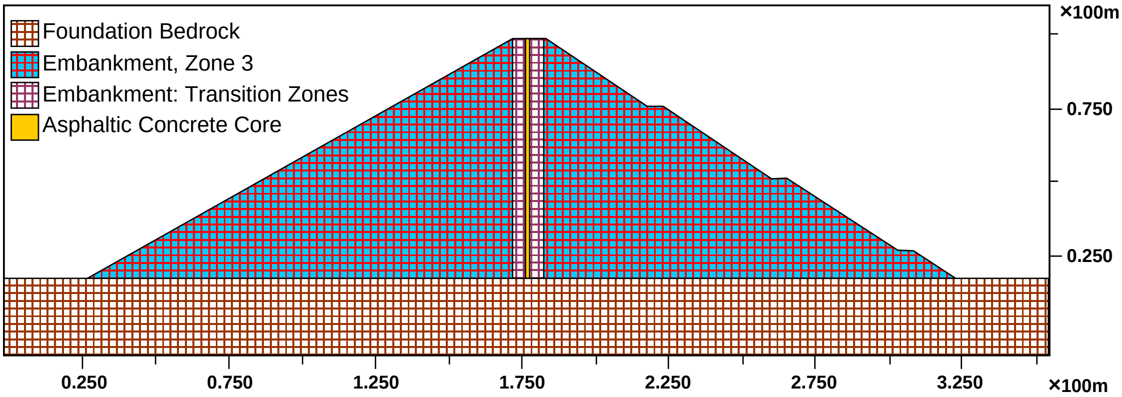

Figure 3: Finite Difference Model of the Dam.

A summary of the deformations predicted by the non-linear dynamic analyses under the three earthquakes are presented in Table 3, where a positive sign indicates movement in the downstream and upward directions. Table 3 also shows the maximum relative displacements of the embankment crest in both horizontal and vertical directions calculated with respect to the top of the bedrock (base of the embankment).

Table 3: Summary of Predicted Displacements from Dynamic Analyses.

Earthquake	Crest settlement (m)			Maximum relative displacement (m)	
	U/S	Core	D/S	Horizontal	Vertical
Loma Prieta	-1.38	-0.19	-0.63	-0.85	-1.44
Nahanni	-1.37	-0.06	-0.39	-1.00	-1.47
Cape Mendocino	-0.75	+0.04	-0.05	-0.60	-0.85

The predicted deformations of the crest are larger in the upstream side of the embankment. The maximum crest settlement predicted by the numerical analysis is 1.38 m, which is approximately 2.5 times larger than the maximum crest settlement obtained using Makdisi and Seed (1978) method. The deformation pattern of the embankment predicted under the Loma Prieta earthquake is shown in Figure 4. This pattern involves settlement and slumping of the rockfill on either side of the core, while the core generally remains vertical. Similar patterns of deformation also resulted from the Nahanni and Cape Mendocino earthquakes.

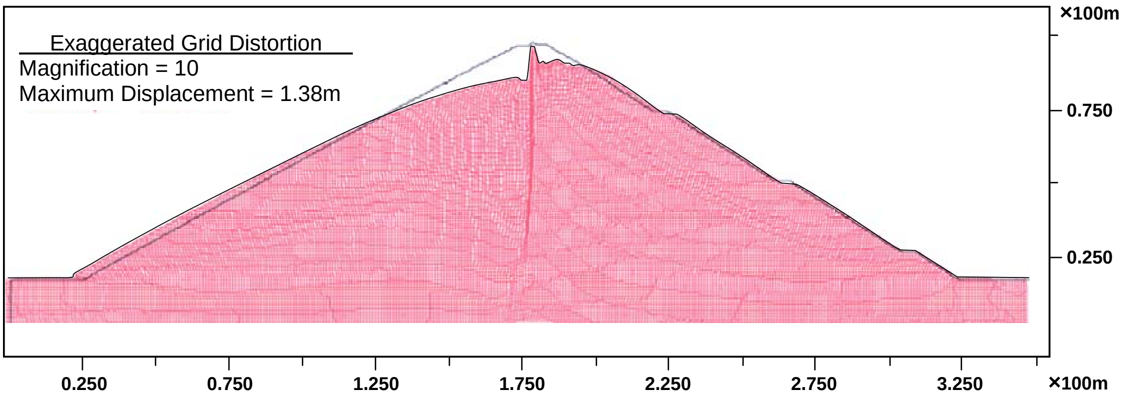

Figure 4: Deformation Pattern Resulted from Dynamic Analysis of the Dam under Loma Prieta Earthquake.

Contours of vertical and horizontal displacements in the body of the dam predicted for the Loma Prieta and Nahanni earthquakes are shown in Figures 5 and 6, respectively. The upstream slope exhibits larger deformations, mainly due to the lower initial effective stresses due to saturation, as also predicted by the pseudo-static analyses. The maximum predicted vertical settlement is at crest level and attenuates rapidly with depth. A maximum horizontal displacement of about 0.8 m was predicted in the lower section of the upstream slope for Loma Prieta earthquake. The Nahanni earthquake resulted in the highest horizontal displacement; about 1.0 m of movement in the middle of the upstream slope.

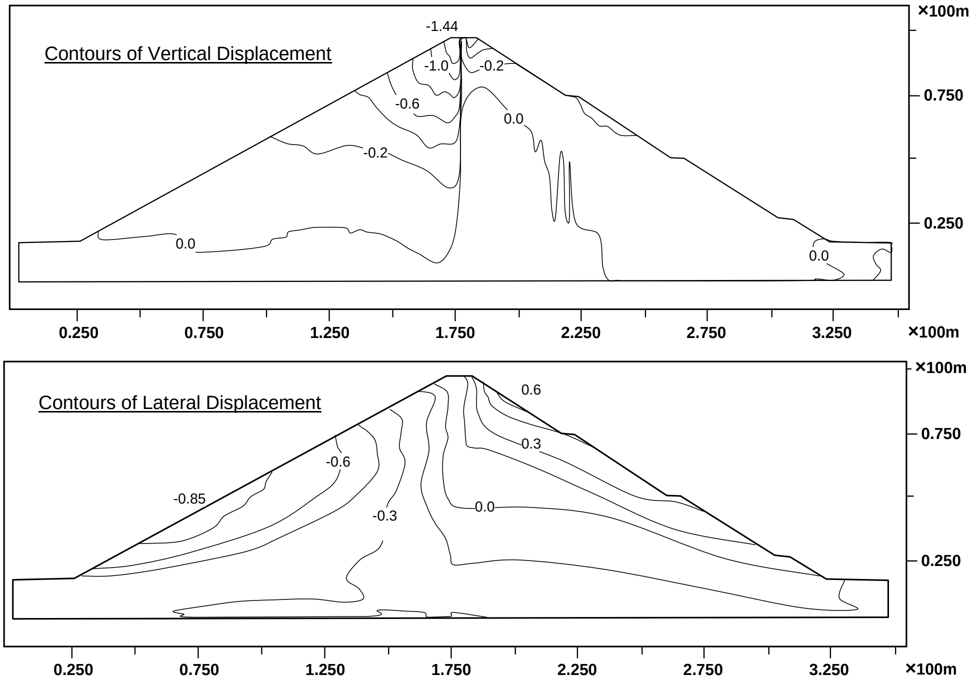

Figure 5: Contours of Vertical and Lateral Displacements under Loma Prieta Earthquake.

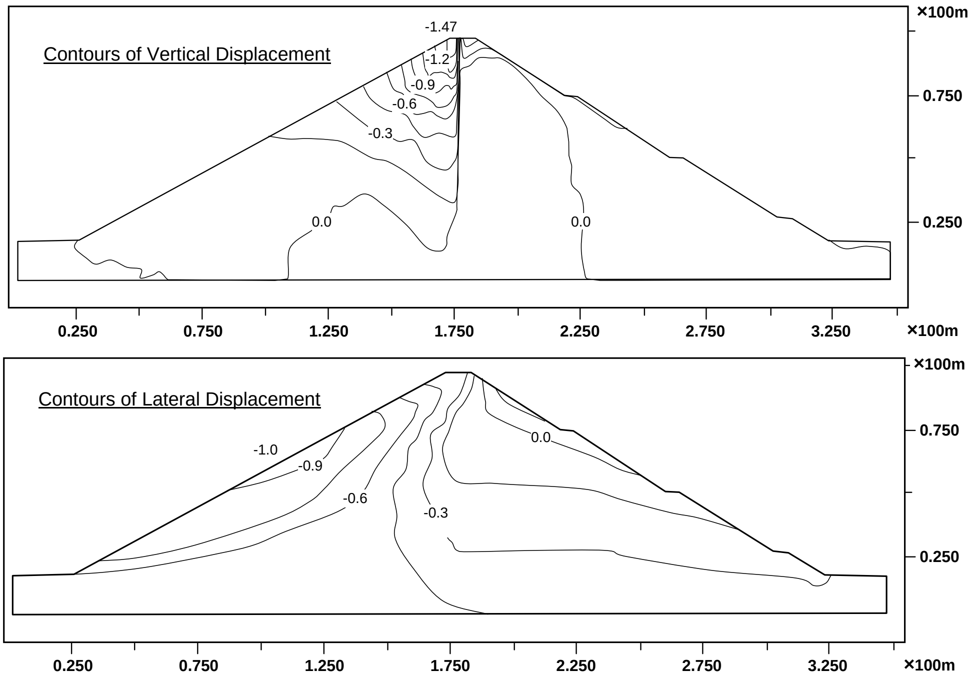

Figure 6: Contours of Vertical and Lateral Displacements under Nahanni Earthquake.

The relative horizontal displacement of the asphalt core, measured with respect to the base of the core, resulting from the analyses under the effects of the three earthquakes, is shown in Figure 7. It can be seen that the maximum differential lateral displacement of the core was predicted to be larger under the Nahanni earthquake with a maximum of 0.4 m in the top quartile of the embankment. However, even at this level of displacement, failure or cracking of the core is not expected.

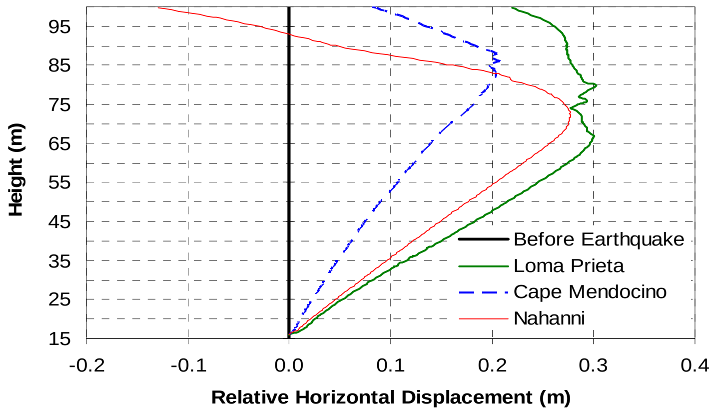

Figure 7: Relative Horizontal Displacement of Asphalt Core.

Variations of the vertical deformations of the crest as a function of time are shown in Figure 8. The predicted peak crest accelerations were about 1.1 g, for Loma Prieta earthquake, 1.4 g for Nahanni earthquake, and 1.7 g for Cape Mendocino earthquake, as compared to the peak ground acceleration of 0.8 g. This indicates an amplification of 1.4 to 2.1. These values compared well with those obtained using the Makdisi and Seed (1978) method. This range of amplifications is expected for rockfill embankments based on past experience, for example Matsumoto *et al*. (2005).

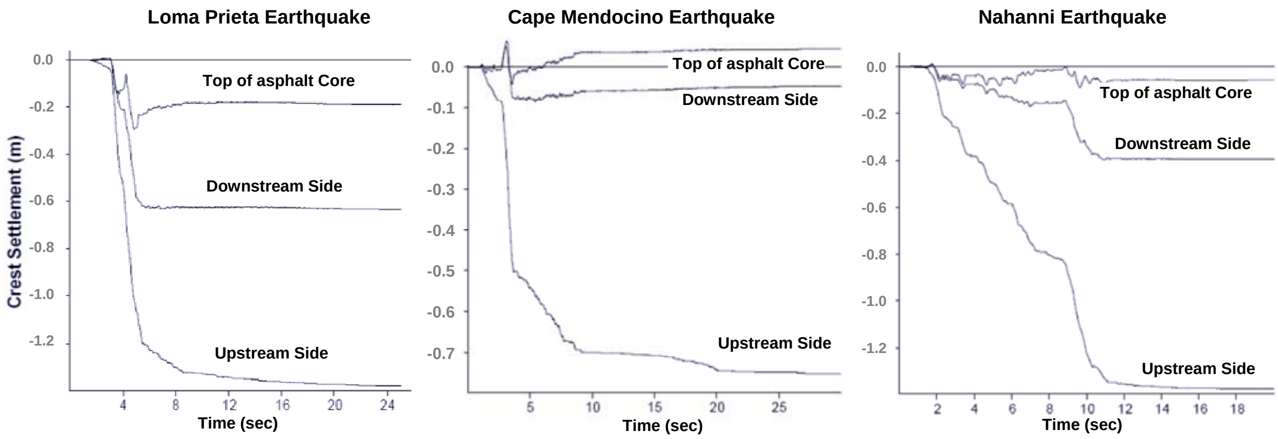

Figure 8: Variation of Vertical Displacements with time under the Three Earthquakes.

The results of the deformation analyses can be compared with observed settlements of embankments subjected to earthquake loading. Swaisgood (2003) examined the observed crest settlements of 69 embankment dams during past earthquakes and provided a mathematical relationship to predict the extent of earthquake induced settlement. This relationship only requires the earthquake magnitude and peak ground acceleration (PGA). Based on the Swaisgood (2003) relationship the crest settlement of Shur River dam is calculated as 3.1% of the embankment height or 2.6 m. This is almost twice the maximum crest settlements predicted in the deformation analysis presented here. Note that the Swaisgood relationship is based on the maximum PGA experienced by the dams in the collected database, none of which exceeded 0.68 g, and therefore requires extrapolation of the observed settlements. Considering the effects of the distance from the earthquake source and the type of the dams reviewed, the maximum predicted displacements from the analysis here would be within the lower boundary of the observed crest settlements in the embankments during past earthquakes, and are not unreasonable.

7 CONCLUSIONS

The results of these analyses indicate that the simplified methods based on Newmark (1965) and Makdisi and Seed (1978) may not necessarily result in a conservative estimate of the crest settlement for rockfill dams as is currently expected in the profession. The magnitude of the crest settlement predicted by the dynamic elasto-plastic modelling is more than two times that predicted by the Makdisi and Seed (1978) method, and about four times that predicted by the Newmark (1965) method. This is in agreement with the findings of Feizi-Khankandi *et al.* (2009). The concept of a unique failure plane, as adopted by the simplified methods, may be misleading, as deformation would be more likely to be characterised by the deformation pattern predicted using a non linear elasto-plastic model and FLAC software.

Under normal operating conditions the Shur River Dam has a minimum freeboard of 4.0m between crest level and spillway level; thus deformations of the predicted magnitude would not cause the storage to be breached and are considered to be acceptable. Based on the predicted deformation of the embankment core and expected level of internal strain in the core material, failure of the embankment core is not expected.

8 ACKNOWLEDGEMENT

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