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ABSTRACT

The paper pertains to the seismic analysis of earth and rockfill dams with the aid of MSC_ Nastran (Windows) package. After validation, the package has been used to investigate the dynamic response of Tehri Dam, located in the seismically active region of Himalayas. A 2D FEM analysis is adopted wherein the dam has been modeled as a linear, elastic, non-homogeneous material. The base acceleration data of the Bhuj Earthquake has been used as an input motion. Effect of Poisson's ratio and the ratio of the canyon length to the height of the dam has been investigated and is reported. Acceleration-time histories reveal that the maximum acceleration occurs at the crest of the dam, and decreases towards the bottom of the dam. Displacement-time histories reveal that the vertical displacement at any locations of the dam is negligibly small compared to the horizontal displacement. The shear stresses evaluated displays a maximum and minimum magnitude at the shell and core of the dam respectively. Velocity-time history shows a maximum velocity in the forward direction at the crest of the dam, while in the reverse direction, the same is experienced by the shell and the core of the dam supplemented by a noticeable phase difference.

INTRODUCTION

Despite considerable advances in the field of earthquake geotechnical engineering, earthquakes continue to cause destruction of life and damage of property. Even though the total duration of earthquakes during this century has been less than one hour, the damages caused are extensive with more than 2 lakh casualties. Recent large scale earthquakes have damaged many agricultural facilities such as canals, farm, roads and earth dams. Earth dams are especially important in terms of disaster prevention since they provide irrigation water and their damage can cause secondary destruction of nearby habitations.

Till recent times, the engineers assumed that the earth dams have an inherent reserve of strength against earthquakes, and no special measures are required to be taken. Even though suitably designed earth dams can withstand considerable seismic activity with slight or limited damage, such dams when subjected to severe shocks (as has been experienced in the last quarter of the previous century) may fail due to the accumulated damage resulted from the superposition of the dynamic forces from successive major earthquakes. Under such circumstances, the failure of such dam may be catastrophic for habitation and agriculture. As such, it is very important that in designing such earth structures, analysis be made considering the possibility that these may experience severe seismic shocks during its service period, so that adequate safety provisions can be made for better performance during earthquake.

Assessment of the performance and stability of earth dams during earthquakes requires a dynamic response analysis to determine the acceleration, dynamic stresses and deformations induced in the dam by the seismic forces. In current engineering practice, the dynamic response of earth dams (located in valleys or narrow canyons) subjected to high-magnitude earthquakes is usually determined by independently computing the dynamic response of the individual sections of the dam by carrying out a 2D-analysis.

The purpose of this paper is to present a 2D-FEM (2 dimensional-Finite element method) analysis of earth and rockfill dams in order to check the suitability of the same in determining the dynamic behavior of such dams.

NATURE OF EARTHQUAKE DAMAGE

A comprehensive summary of the known earthquake damages to 58 earth dams was prepared by Ambraseys (1960), and is briefly reported as follows.

- Langley Dam (USA, 1886, Eq. Intensity-7): A number of cracks resulting in leakage and destruction of greater part of the structure.
- Greggs Dam (USA, 1886, Eq. Intensity-9): Extensive longitudinal cracking and slides on both slopes, resulting in settlement and subsequent overtopping.
- Mineyama Dam (Japan, 1886, Eq. Intensity-9): Settlement and longitudinal cracks along crest.
- Idu Dam (USA, 1930, Eq. Intensity-5): Failure of the outlet pipe and consequent internal erosion and failure of dam with loss of life.
- Usantho Dam (Formosa, 1930): Slide of the u/s slope, bulging and slides of lower half of d/s toe.
- Hunakawa Dam (Japan, 1939, Eq. Intensity-5): Washed out completely.
- Oy Dam (Japan, 1945, Eq. Intensity-5): Outlet pipe placed in trench in the gravel badly damaged, some spreading of the fill.
- Otani Dam (Japan, 1946, Eq. Intensity-5): A longitudinal crack 75 m long at crest and others 10-50 m on u/s slope – apparently an incipient slide.
- Ogawa Dam (Japan, 1946, Eq. Intensity-5): Heavy leakage near one of the abutments, subsequently repaired by cement grouting.
- Nagi Dam (Japan, 1946, Eq. Intensity-4): Leakage at right abutment, fracture of outlet.
- Shiote Dam (Japan, 1946, Eq. Intensity-4): Slumping and cracking in d/s slope.
- Nichiman Dam (Japan, 1946): Presumed cracking and leakage.
- Mizusako Dam (Japan, 1946, Eq. Intensity-4): Slight damage to intake conduit.
- Sakura Dam (Japan, 1946, Eq. Intensity-5): Cracking.

Table 1 provides a brief enlistment of the number of earth dams damaged in various earthquakes.

The Hyogo-ken-nainbu earthquake damaged 1222 earth dams. While only eight were completely destroyed, the others caused havoc secondary damage to the downstream urban localizations.

Table 1. Earth dams damage caused by several earthquakes

Earthquake	Time of Occurrence	Magnitude	No. of earth dams damaged
Kita-tango	1927	7.5	90
Oga	1939	7	74
Niigata	1964	7.5	146
Matsushiro	1965	5.4	57
Tokiachi-oki	1963	7.9	202
Miyagi-kem-oki	1978	7.4	83
Nihon-kai-chubu	1983	7.7	238
Chiba-kentoho-oki	1987	6.7	9
Hokkaidonasei-oki	1993	7.8	18
Noto-hanto-oki	1993	6.6	21
Hyogo-ken-nainbu	1995	7.2	1222
Ishikari-hokubu	1995	5.6	1
Kagoshima hokuseibu	1997	6.1	1

GUJARAT EARTHQUAKE (26TH JAN, 2001)

On 26th January, 2001, a powerful earthquake struck in the Bhuj region in the state of Gujarat, located in the western part of India. Department of United States Geological Survey (USGS) classified the magnitude of the earthquake as 7.9, which was later routinely downgraded to 7.7. Intra-plate collision was attributed as the prime factor to initiate the earthquake in Gujarat, the epicenter of the quake being located 200 km from Ahmadabad. The earthquake caused maximum damage to the district of Bhuj. In Bhuj, almost no building had been left standing. Five of the many earth dams damaged in the earthquake are shown in the following photographs [Figure 1].

Figure 1a: Vertical and Crest settlement of Suvi Dam

Figure 1b: Longitudinal cracks in the crest of Fategarh Dam

Figure 1c: Longitudinal cracks in the crest of Tappar Dam

Figure 1d: Upstream slope failure of Fategarh Dam

Figure 1e: Massive cracks in the upstream embankment near the toe of the Fatehgarh dam

Figure 1f: Longitudinal cracks in the crest of Kaswati Dam

Figure 1g: Lateral spreading in the toe of Kaswati Dam

Figure 1h: Cracking and Slumping in the upstream face of Rudramatha Dam

From a study of earthquake damage to these earth dams, it can be observed that cracking and crest settlement are the most common types of damage. Cracking may often result in leaks, which may develop rapidly causing subsequent failure. Soft foundation or those susceptible to liquefaction may result in serious damage or failure. Large landslides into the reservoir or near dam abutment may result in damage by overtopping and piping respectively. Slope slides have taken place in several cases, though few have resulted in complete failure. More serious landslides have, however, taken place in natural slopes *e.g.* Anchorage, Alaska in 1964. Movements on faults may cause the rupture of the dam raisings.

FAILURE OF EARTH DAMS DUE TO EARTHQUAKES

The possible ways of the failure of earth dams due to earthquakes have been enlisted by Sherard *et al.* (1963) that are stated as follows: (a) Failure due to disruption of the dam by major fault movement in the foundation (b) Slope failure induced by ground motions (c) Loss of freeboard due to differential tectonic ground movement (d) Loss of freeboard due to slope failure or soil compaction induced by ground motions (e) Piping failure through cracks due to ground motions (f) Overtopping of dams due to slides or rock-falls into the reservoir (g) Sliding of dams on weak foundation materials, and (h) Failure of spillway or outlet works.

Adequate design precautions should be adopted to preclude any possibility of failure due to the above causes and often simply involve the exercise of good planning and judgment along with the incorporation of the features such as (a) Avoidance of active faults in the foundation (b) Provision of ample freeboard to allow for some loss due to subsidence of slope slumping (c) Provision of wide transition section of filter materials that are not vulnerable to cracking (e) Use of such materials in wider core that are capable of self-healing if there be any eventual development of cracking (f) Careful examination of the stability of slopes adjoining the reservoir, and (g) Provision of appropriate crest details to minimize erosion in the event of overtopping.

Such design measures might provide adequate protection against all the detrimental effects of earthquakes except basal sliding or slope failures. The possible development of slope failures is customarily evaluated qualitatively, based on experience, and quantitatively based on the analysis carried out with the aid of the available analytical techniques.

PSEUDO-STATIC ANALYSIS

If the peak acceleration of the ground motion is known, the inertial force on the soil element can be obtained by multiplying the mass of the element with the seismic acceleration coefficient. If this force is then accounted for determining the equilibrium of the potential sliding mass, and a factor of safety of more than one is prescribed, it implies that no movement of the sliding mass is allowed and limit equilibrium has been maintained. Such an approach is called the pseudo-static analysis, wherein the dynamic effect of the earthquake is replaced by a pseudo-static force, and the equilibrium is maintained inclusive of this force. During an earthquake, the acceleration in a particular direction reaches a maximum and then reduces to zero, and then subsequently the whole cycle is reversed; the duration of which limits from a fraction of a second to few seconds. Further, the dam is subjected to only a limited number of cycles, amongst which a few number of cycles reaches peak acceleration. Thus, the adverse inertial force lasts for a very short interval in each cycle, and persists only for a limited number of cycles. The deformations caused due to such a phenomena depend on the effective duration. The pseudo-static approach fails to distinguish between the short-duration inertial force and the static forces of much longer duration. In this approach, the horizontal seismic force is accounted by multiplying the weight of the slice [W] by a horizontal seismic coefficient [K_Y], and hence, the horizontal seismic force is accounted as [$K_Y.W$]. K_Y can be estimated by using any of the several methodologies developed by Bishop (1955), Morgenstern and Price (1965), Janbu (1973) and Spencer (1978) to calculate. Seismic stability of a dam can be found out with some confidence using such an analysis if the expected earthquake acceleration is much less than the critical acceleration of the dam.

Rigid Body Response Analysis

A rigid body response assumes a coefficient equal to the maximum ground acceleration acting along the entire height of the dam. However, both the theoretical and field studies indicate that the dams do not behave as rigid bodies. Neglect of the viscous damping and short duration force leads to highly conservative design; while the assumption of constant seismic coefficient with height may lead to the unsafe design of the top portion of the dam.

The main limitations of this approach are as follows: (a) Although low, stiff embankments located in narrow canyons may respond essentially as rigid structures, considerable evidence based on field studies (in which the actual dams have been subjected to forced vibrations by means of large shaking machines) reveal that the earth dams studied do not behave as rigid bodies but respond in different ways to any given series of imposed shocks. (b) The maximum acceleration will only be developed in an embankment for a short period of time so that the deformation resulting from it may be small. Although it will be supplemented by deformation produced by other acceleration and inertia forces developed during the earthquake, there is no reason to believe that their combined effect will be equivalent to those proposed by applying inertia force, corresponding to the maximum acceleration in the embankment as a static force, *i.e.*, as if it were acting for an unlimited period of time.

Use of Empirical Values

A more common practice has been to assume an empirical seismic coefficient based on prevalent practice for design of rigid dams. A value of 0.1 – 0.15 has been commonly adopted in USA, while the Japanese earth dam code requirement prescribes higher values from 0.15 – 0.25. In India, Ramganga dam has been designed for a seismic coefficient of 0.12, while a value of 0.15 has been considered for the Beas dam. This practice of using empirical coefficient has no fundamental basis. Using a coefficient of 0.1 – 0.15, along with allowing the factor of safety to go down from 1.5 (under static condition) to just over unity (under dynamic condition is a common practice.

Using Gutenberg and Richter's (1956) equation, Jai Krishna (1962) has plotted curves estimating the probable maximum anticipated ground acceleration. Housner (1959) has provided curves based on actual measurement of response to earthquakes in California, USA. By using a suitable factor in accordance to the maximum ground acceleration, the response for the top of the dam has been determined. Use of Housner's spectral curves for working out the dynamic response of an earth dam to specified ground motion is rather approximate since the spectral curves were calculated for single mass system from the recorded acceleration of a point on the ground whereas the base of an earth dam may be several hundred meters in both directions. The effective back and forth acceleration will be somewhat less than that of a point for short period seismic waves with wavelength much smaller than the base.

A procedure suggested by Housner to apply the spectral curves to distributed mass system has been discussed by Sharma (1964). The standard method of slices is easily adoptable to the use of a variable seismic coefficient. Seismic coefficient can be taken from Hatanaka's solution (1955). For each slice, the values of horizontal and vertical forces can be computed according to the values of K_Y, which can then be added up for all the slices to compute the factor of safety. It should be noted that with variable K_Y, the critical circle itself may be changed. It is quite likely that the critical circle may no longer be passing through the toe, but may encompass only a part of the slope near the top. Therefore, it is highly necessary to check the slip surface that does not pass through the toe of the dam. Seed (1967) demonstrated that the analytical details of the stability computation usually outweigh the small variations in the values of the seismic coefficient. Computations during the preliminary design studies of the Oroville Dam revealed that varying the details of the analytical procedure could easily change the seismic coefficient from 0.1to 0.23, retaining a constant value of factor of safety equal to 1.1.

Limitations of Pseudo-Static Analysis

Based on previous experience, pseudo-static method of analysis can provide some guidance in evaluating the slope stability during earthquakes. However, their use is quite limited due to the lack of proper knowledge. The pseudo-static method of analysis incorporating seismic coefficients of the order of 0.1 fails to explain the field cases of slope failure that are studied in detail. Three major slope failures in Anchorage occurred by sliding near the surface of a layer of soft, sensitive clay; however, a pseudo-static analysis of the studies indicated that the failure would develop at the base layer. A pseudo-static approach also fails to explain the time delay in the initiation of the failure resulting from the start of the ground motions. Since the pseudo-static approaches fail to provide a reasonable evaluation of the slope behavior in few of the well-defined case studies, their usefulness must be considered to be limited to regular practice. Moreover, the selection of the design seismic coefficient to be used in the pseudo-static analysis is largely dependent on the method of analysis for which it is to be used, and is difficult to assume a proper value for the coefficient.

DEFORMATION ANALYSIS / LIMIT EQUILIBRIUM DESIGN

The basic concept of the limit equilibrium design is that during an earthquake no non-elastic deformations should take place at all, and the factor of safety should remain unity at all times. Since the adverse earthquake forces usually last for only a few cycles of very short durations, the limit equilibrium approach is rendered unrealistic for the embankment dam analysis. Deformation takes time to develop, and even if the average shear resistance falls below the shear stress on a section through the embankment, very small movement takes place in the short duration for which the adverse condition persists. The reduction and then the reversal of the acceleration will arrest the movement till the next adverse cycle. If the nature of the soil material is such that there is no significant long-term reduction in shear resistance even if non-reversible deformation takes place, the criterion for design should then be the allowable deformation rather than the limiting equilibrium. The necessity for such a method has also been clearly shown by the observed behavior of the soil during the Alaskan Earthquake of 1963 that revealed that the soil strength mobilized was a function of the entire time-history of the stress developed during the earthquakes. This approach was first proposed by Newmark (1965). This approach has been successfully used to predict the surface displacement of banks of dry, cohesionless soils in which the pore pressures develop as a result of shear strain induced by the earthquake.

The displacement analyses can provide approximate idea of probable deformation of cohesionless material under conditions in which appreciable change in pore pressure does not occur during an earthquake. Thus, in cases where shells of a dam consists of granular material, either unsaturated or freely draining, these procedures should give the order of earthquake displacement. However, in case of saturated non-cohesive material in which pore-pressure built-up may take place during an earthquake and in case of cohesive soils, no suitable analytical method is available and an estimate of displacement can only be made based on laboratory tests. A procedure for doing so has been suggested by Seed and Martin (1966).

Seed and Martin Method for Deformation Analysis (1967)

As suggested by Seed and Martin (1967), the method of deformation analysis is based on the determination of the stresses acting on soil elements within an embankment both before and during an earthquake. Typical soil samples are prepared and are subjected to the same sequence of stress change experienced by corresponding elements in the field and the resulting deformation is recorded. This is then used for the estimating the deformation of the slope from the observation of the comprising soil elements. The method thus considers the following: (a) the time history of forces developed in the embankment or slope during an earthquake, (b) the behavior of the soil under simulated earthquake loading condition, and (c) the desirability of evaluating embankment deformation rather than a factor of safety.

NEW CONCEPTS IN SLOPE STABILITY ANALYSIS

The design of earth structure to safely withstand the destructive effect of earthquakes constitutes a complex analytical problem. During an earthquake, the inertia in certain zones of an embankment may be sufficiently large to drop the factor of safety below unity a number of times, but only for brief periods of time. During such periods, permanent displacement will occur, but the movement will be arrested when the magnitudes of the acceleration decreases or is reversed. The overall effect of a series of large but brief inertia forces may well result in a cumulative displacement of a section of the embankment. However, once the ground motion generating the inertia forces have ceased, no further deformation will occur unless there has been a marked loss in strength of the soil.

Thus, the magnitude of deformation that develops will depend on the time-history of the inertial forces and a logical method of design requires (a) the determination of the variation of the inertial forces with time, and (b) an assessment of the embankment deformation induced by these forces.

Newmark (1963) proposed the concept that the effects of earthquakes on embankment stability should be assessed in terms of the deformation they produce, rather than based on the minimum factor of safety. Based on this concept, Newmark (1965) and Seed (1966) have presented analytical methods to evaluate the effect of earthquakes on the stability of embankments.

It has also been recognized that the soil strength mobilized during earthquakes may be quite different from that determined under static transient loading conditions, and thus the mobilized strength should be regarded as a function of the entire time-history of stress developed during an earthquake.

IMPROVED PROCEDURES FOR EVALUATING SLOPE RESPONSE TO EARTHQUAKES

In keeping with the foregoing concepts, new procedures have been proposed for evaluating the response of embankments and slopes to earthquake ground motions. The developments have been made possible largely through the development of numerical methods (FEM) and the availability of computers for making the detailed computations.

Shear Deformation Theory

Hatanaka's solution (1952) for elastic response assumes idealized sinusoidal oscillations of the ground without considering the actual time history of a typical earthquake. Seed and Martin (1966) devised a method to use the shear slice approach to determine the entire time-history of accelerations and stresses developed during the periods of significant ground motion. In this approach, the average seismic coefficient is defined as follows:

$$K_{av} = (1/W)\sum\left\{M_{(Y)} \times U_{a(Y)}\right\} \qquad (1)$$

where, $M_{(Y)}$ is the mass of an incremental slice of the dam, and $U_{a(Y)}$ is the corresponding absolute acceleration of the slice at the instant under consideration. On this basis, it is possible to express the stress developed at different sections of an embankment in terms of dynamic seismic coefficients. It is also possible to represent the results for embankments of different heights and materials subjected to any arbitrary base motions. The values of the seismic coefficients increase with the increasing elevation of the potential sliding mass within the body of the embankment and also vary with the height of the embankment and the material characteristics. They would also vary with the nature of the earthquake ground motions. This type of information provides the necessary basis for analysis of deformations and for planning of laboratory test procedures.

However, the above method is also not free of limitations such as (a) No vertical ground motions are introduced, (b) Only shear modes are considered, and (c) Shear deformation theory does not permit an accurate picture of the stress distribution within an earth dam during an earthquake.

Finite Element Approach

The dynamic response of an earth dam has been studied by shear deformation theory based on assumption that the dam is comprised of a series of infinitely thin horizontal slices connected by linear elastic shear spring and viscous dashpots. Although the theory provides an indication of the effects of dynamic response, it is limited in its application and does not present an accurate picture of the stress distribution within an earth dam during an earthquake. It is possible to overcome these difficulties with the aid of finite element method wherein no assumption is made regarding the type of deformation or stress distribution. The variation of the soil properties in different zones of the dam can also be taken into account, which is not possible in shear wedge analysis. Seed and Martin (1966) illustrates the first three modes of vibration of a typical earth dam as obtained by the finite element method [Figure 2]. Comparison of the first mode of vibration with those given by the shear deformation theory shows that only a small difference in horizontal displacement occur on the vertical axis, whereas displacement in the dam slopes differ considerably - the finite element displacement being smaller and including an anti-symmetrical vertical component. Occurrence of such reduced displacement in the slopes has been confirmed by forced vibration tests on existing earth dams. In addition to shear mode, finite element analysis introduces vertical and rocking modes. In the vertical modes, as shown in the second mode, the deformation pattern comprises of symmetrical, vertical and horizontal displacements, while in the rocking modes as shown in the third mode, the deformation pattern comprises of anti-symmetrical, vertical and horizontal displacements. In general, the shear modes given by finite element analysis have vertical frequencies approximately 10% less than those given by shear deformation theory.

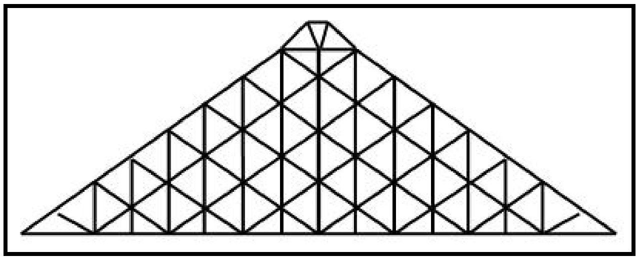


Figure 2a: Finite Element Discretization

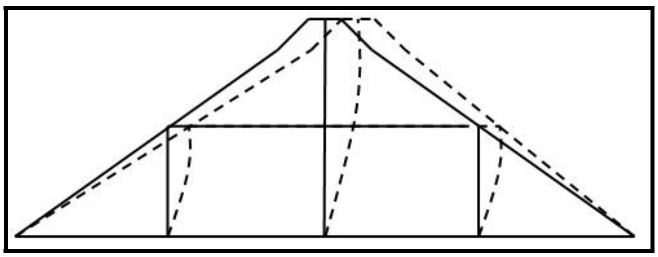


Figure 2b: 1st Mode (Shear)

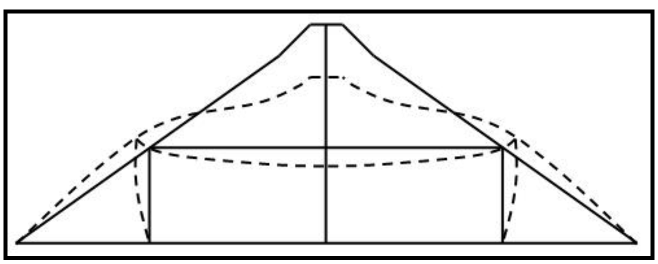


Figure 2c: 2nd Mode (Vertical)

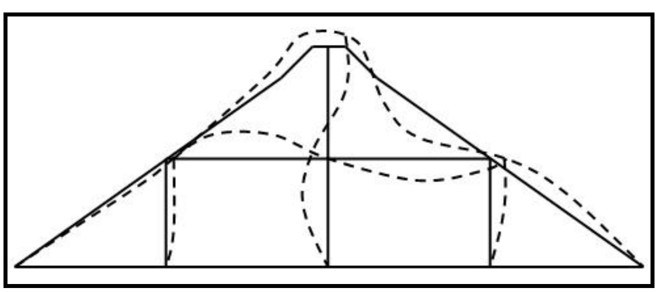


Figure 2d: 3rd Mode (Rocking)

Preliminary earthquake response analysis using the finite element theory has shown that the dynamic shear stress and acceleration distribution over the horizontal planes are not particularly uniform; values in the vicinity of the vertical axis are greater than values near dam slopes. However, the magnitude of the uniformly distributed shear stress and lateral acceleration given by the one-dimensional uniform distribution are more than that given by the finite element theory. Further, stress contours evaluated at various instant during the earthquake have indicated that extensive zones of tension can develop on the slopes of the dam. These tension zones could lead to surface cracking and such cracks have been observed on the slopes of earth dams exposed to earthquakes. The finite element analyses have also indicated that the vertical acceleration induced in the dam by both the vertical and horizontal ground motion could have significant magnitudes. It is evident that the finite element method is a powerful technique for investigating the earthquake response of the earth dams [Clough and Chopra (1966)].

PROBLEM STATEMENT

From the above discussions, it is realized that the determination of seismic stability of an earth dam usually involves a dynamic response analysis of the dam for the maximum earthquake motions likely to affect the structure. FEM is one of the most versatile and useful tool for analyzing and solving such problems. Clough and Chopra (1965) carried out significant research on the application of FEM to determine the dynamic response of earth dams. Hatanaka (1967) and Ambraseys and Sarma (1967) studied the response of earth dams. Vrymoed (1981) proposed a technique based on adjusted soil properties to study the three-dimensional response of an earth dam using two-dimensional FE technique. This paper deals with the dynamic analysis of earth and rockfill dams (Tehri Dam) subjected to Bhuj earthquake ground motion (described previously). An earthquake [namely the Kangra earthquake of such a large magnitude (Magnitude 7.7)] occurred in 1905 in the vicinity of the dam. However, the earthquake spectrum of the same is not available. Therefore the spectrum for the Bhuj earthquake having similar magnitude is used to analyze the dam. The dam has been modeled as a linear, elastic and non-homogeneous material and FEM has been used to carry out a 2D plane strain analysis. Effect of the Poisson's ratio and the ratio of the canyon length to the height of the dam has been investigated. MSC Nastran package has been used for this purpose.

Details of Tehri Dam

The dynamic response of the Tehri Dam is determined by carrying out a 2D-FEM analysis. Tehri Dam is located at new Tehri town, Tehri District, Uttar Pradesh (now in Uttaranchal State), and is constructed upon river Bhagirathi. The dam is primarily meant for hydropower in the seismically active Himalayas, a region that is expected to experience an earthquake of magnitude 8.0. The salient data of the project are as follows: Tehri dam is an earth and rockfill dam and it is constructed on slightly jointed phyllites of different grades. Maximum height of the dam is 261 m, and the length of the dam is 570m. Total area of the reservoir in Full Reservoir Level (FRL) condition is 42 km^2, and the gross storage capacity is 3539 Mm^3. The cross-section of the dam is shown in Figure 3.

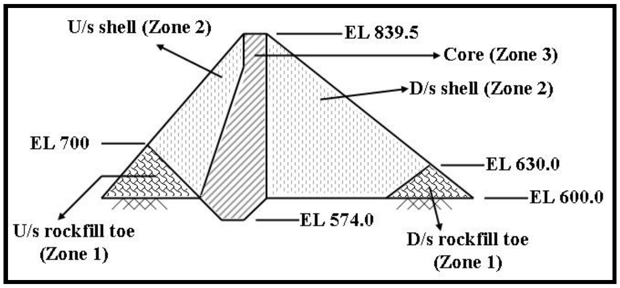


Figure 3: Cross-section of Tehri Dam

<u>Plane Strain Problem-2D FEM Analysis</u>

Considering that the conditions of plane strains are satisfied, a 2D FEM analysis of the Tehri Dam is carried out. For analysis purpose, the bottom of the dam is considered to be fixed. Four-noded quadrilateral and triangular elements have been used. A total number of 595 elements are considered. As already stated Bhuj earthquake (26th Jan, 2001, India) have been considered for dynamic transient analysis of the dam. It is assumed that both the u/s and d/s rockfill dam (Zone 1) consist of boulders with average density of 2250 kg/m^3, Modulus of elasticity (E_R) = 1.02e8 N/m^2, Poisson's ratio (ν_R) = 0.34. The shell of the dam (Zone 2) is considered to be consisting of gravel and sand mix with a density of 2100 kg/m^3, E_S = 9.4e7 N/m^2, ν_S = 0.36. The core of the dam (Zone 3) comprises of silty-clay with a density of 1950 kg/m^3, E_C = 3e7 N/m^2, ν_C = 0.45. Figure 4 depicts the discretization of the dam.

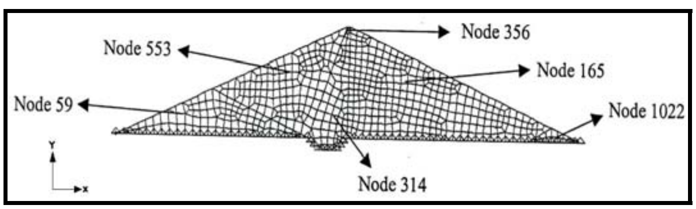


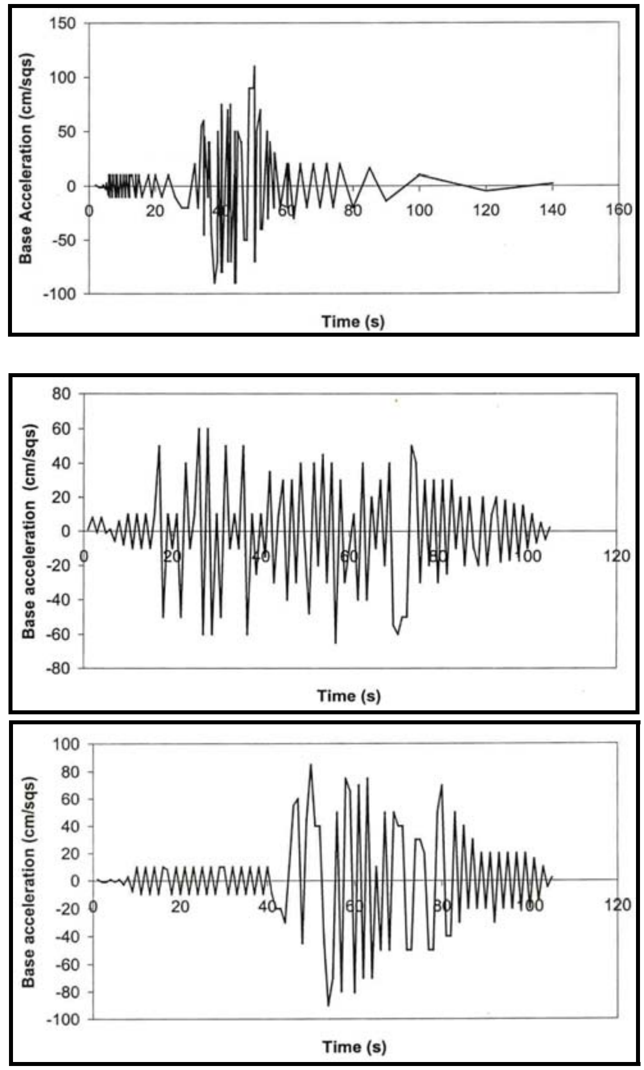
Figure 4: Discretization of 2D model

<u>Input Motion</u>

Base acceleration data at Ahmadabad due to Bhuj earthquake (26th Jan, 2001) is shown in Figure 5. Amongst these, the vertical and the N-S component has been used to carry out the 2D analysis.

RESULTS AND DISCUSSIONS

With the above inputs, a 2D-FEM analysis of the earth and rockfill dam (Tehri Dam in this case) has been carried out with the aid of a software package (MSC Nastran-Windows). Validity of the MSC.Nastran package has been verified with the results obtained from the analysis of Oroville Dam. The effect of Poisson's ratio on the displacement of the dam has also been studied and reported.



(*a*)
(*b*)
(*c*)

Figure 5: (a) N-S component (b) Vertical Component, and (c) E-W component of base acceleration data from Bhuj earthquake, Ahmadabad (26th Jan, 2001)

<u>Calibration of the Package</u>

To check the suitability of MSC.Nastran and accuracy of the results, a calibration test has been carried out with a solved example of the Oroville Dam by Vrymoed (1981). Oroville dam of the California State Water Project is situated in the foothills on the western slope of the Sierra Nevada. The dam is built across river Feather, and is the highest earth-fill dam in the United States. It rises 235 m above the streambed excavation and spans for 1700 m between the abutments at the crest. The dam consists of three zones: Zone 1 consists of impervious core consisting of well graded mixtures of clays and silts, Zone 2 and Zone 3 consists of the shell made from the sands, gravels, cobbles and boulders. 3D-FEM analysis has been used wherein 8-noded brick element and 6-noded tetrahedral element were chosen. An overall damping of 5% has been used. Oroville earthquake (1st Aug, 1975) has been used for the analysis, the corresponding acceleration time history is shown in Figure 6.

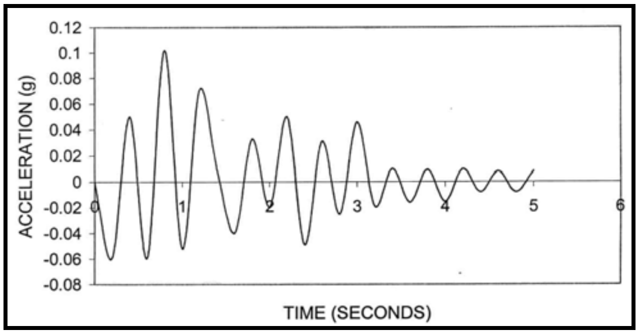


Figure 6: Observed acceleration-time history of Oroville dam

The material properties (conforming to the Oroville Dam) chosen for the analysis are as follows: $E_S = 9.3e7$ N/m^2, $\nu_S = 0.35$, $\gamma_S = 23.5$ kN/m^3, $E_C = 2.5e7$ N/m^2 and $\nu_S = 0.45$. It has been observed that the present study has an excellent agreement with results obtained by Vrymoed (1981), as is evident from Figure 7.

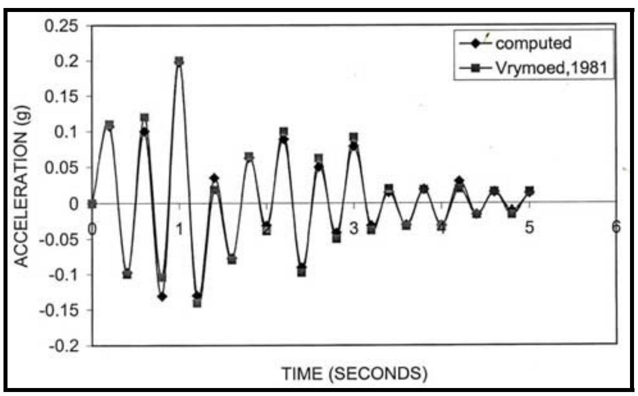


Figure 7: Computed acceleration at the crest of Oroville dam

Plane Strain Analysis

Two-dimensional Finite Element Analysis has been performed for the critical cross-section of the Tehri Dam. Modeling and material properties chosen for the analysis have been stated in the earlier section. Bhuj earthquake (26th Jan, 2001) have been used as the input motion.

Based on the past literature, the most appropriate parameters that can be chosen as variables in an earth dam analysis are accelerations, velocities, displacements, shear stresses and the ratio of shear stress to the vertical normal stress. Several case studies [Seed *et* al. (1969, 1973) and Makdisi *et* al. (1978] revealed that for plane strain conditions, the estimation of the shear stress values on horizontal planes (τ_{XY}) (that control the generation of the pore pressure within the earth core and the deformation of the dam materials) provides a satisfactory assessment of the seismic stability of earth dam.

Figure 8 depicts the acceleration-time history at different points of the dam for plane strain analysis. It is observed that the maximum acceleration (0.105g) occurs at the crest of the dam. The accelerations in the dam decreases considerably towards the bottom of the dam. Figure 9 shows the displacement-time histories for different points of the dam obtained from the plane strain analysis. Maximum horizontal displacement of 6.1 mm is observed to occur at the crest of the dam. Vertical displacements at different points in the dam, as revealed in Figure 10, are found to be negligible in magnitude as compared to the horizontal displacements.

From Figures 11 and 12, it can be observed that for the plane strain analysis, the shear stress in the XY-plane along the bottom of the dam and the maximum value of shear stress at different points in the dam are approximately 74 and 68 kPa. Figure 11 reveals that the shear stress is maximal at the shell of the dam, while the core experiences minimum shear stress. Figure 13 depicts the ratio of shear stress to vertical normal stress [τ_{XY}/σ_V] along the bottom length of the earth and rockfill dam. The ratio is maximum (0.8) along the bottom length of the earth and rockfill dam. Figure 14 reveals the velocity-time history for the different points of the dam. The figure shows that velocities are also maximum (1m/s) in the forward direction at the crest of the dam. Both crest and shell experiences similar movement (1m/s) in the reverse direction. However, a phase difference is observed between the occurrences of such velocities.

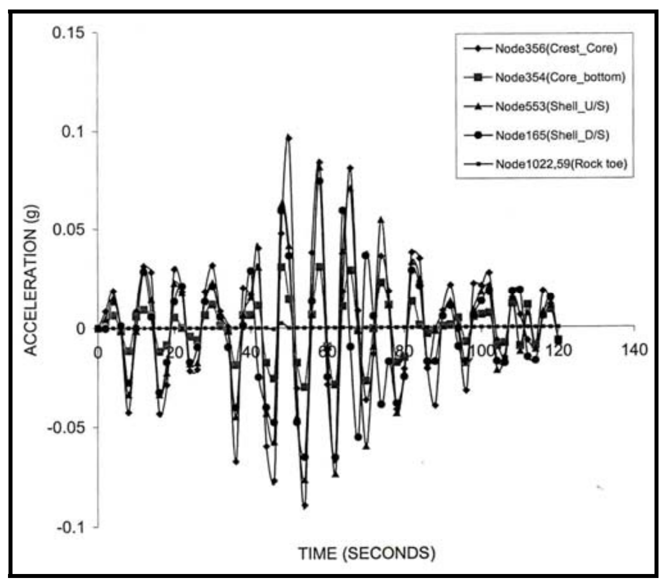


Figure 8: Acceleration at different nodes of the dam

Table 2 provides the magnitude of the computed displacements, velocities and accelerations. Table 3 enumerates the magnitudes of shear stresses and the ratio of vertical normal stress to the shear stresses for the 2D plane strain analysis

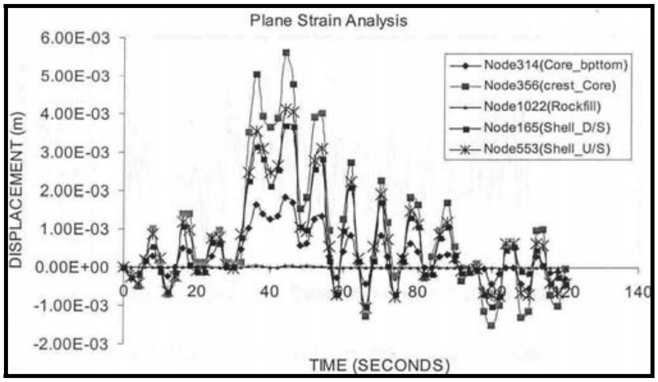


Figure 9: Displacement at different points of the dam

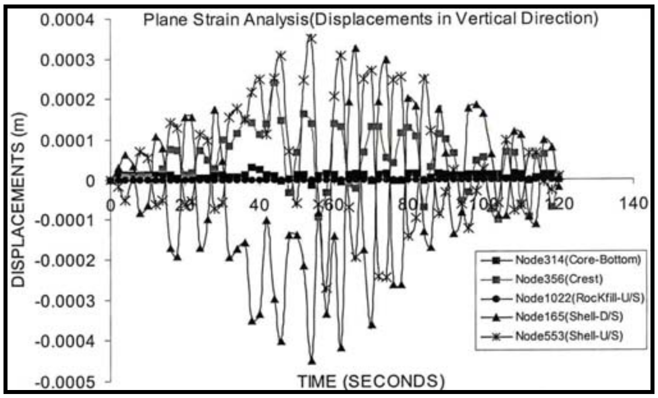


Figure 10: Displacement in vertical direction at different points of the dam

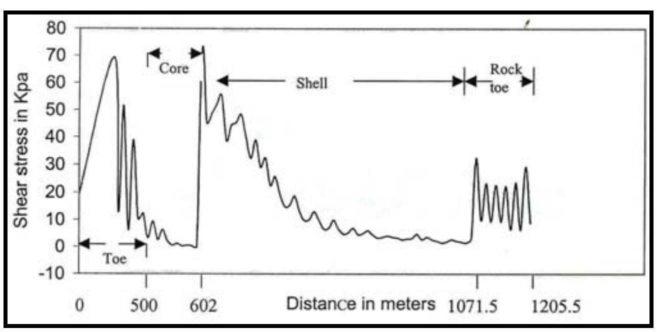


Figure 11: Shear stress along the bottom of the dam

Table 2. Computed displacements, velocities and accelerations

Parameters	Crest	Bottom
Acceleration	0.105g	0.039g
Displacement (mm)	6.1	1.99
Velocity (m/sec)	1.0	0.4

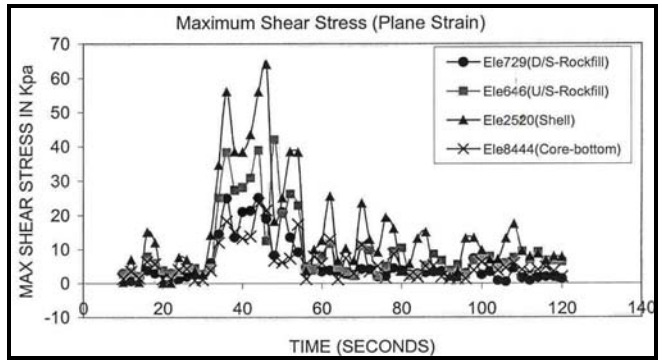


Figure 12: Maximum shear stress for different elements

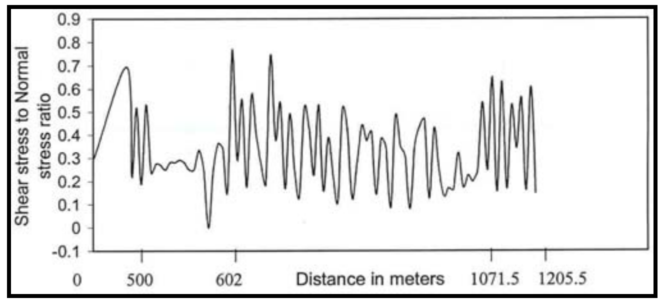


Figure 13: Ratio of shear stress to vertical normal stress along the bottom of the dam

Table 3. Shear stresses and ratio of shear stress to vertical normal stress along the bottom of the dam

Location	Max. shear stress [$(\sigma_1-\sigma_2)/2$] (kPa)	Max. shear stress [τ_{XY}] (kPa)	Max. value of τ_{XY}/σ_V
U/s toe	45	69	0.7
Core	16	12.5	0.32
Shell	65.2	75	0.8
D/S Toe	28.5	30	0.65

Effect of Poisson's Ratio of the Soil

Figure 15 depicts the variation of acceleration with time for the node (356) located at the crest of the dam. The Poisson's ratio is considered to be 0.49 and 0.4 for undrained and partial drainage case respectively. It is observed that the Poisson's ratio does not affect the acceleration of the crest significantly when the time is less than 30 seconds and greater than 100 seconds. The maximum effect is seen at about 55 seconds, wherein the Poisson's ratio is increased to a magnitude of 0.49 from 0.4 and the values of acceleration are decreased from 0.17g to 0.15g, the percentage reduction being 11.5%. This phenomenon is observed due to the weakening of soil with the passage of time due to vibration (when the acceleration at the crest attains its peak). A higher Poisson's ratio provides greater stiffness to the soil against vibration and reduces the acceleration at the crest of the dam in a later stage of vibration.

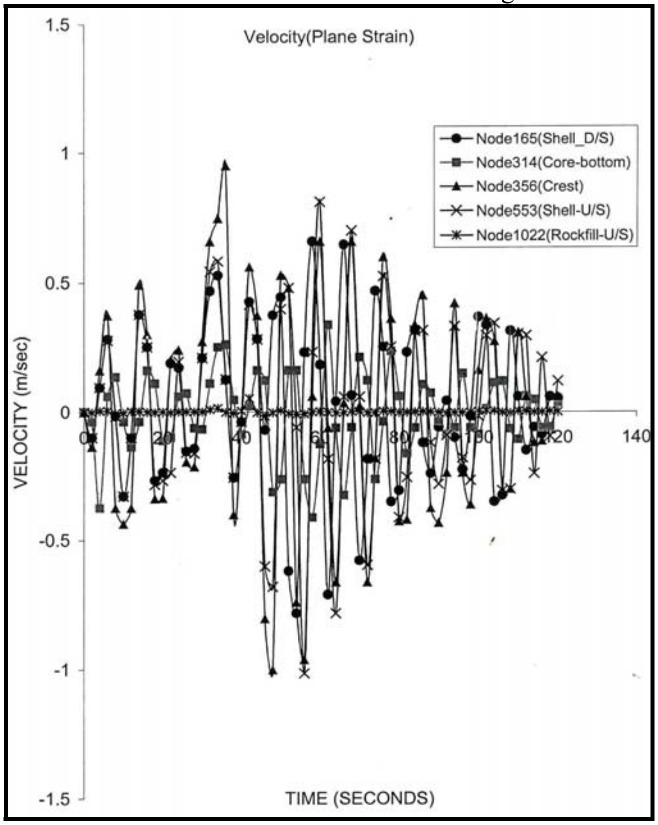


Figure 14: Velocity at different points in the dam

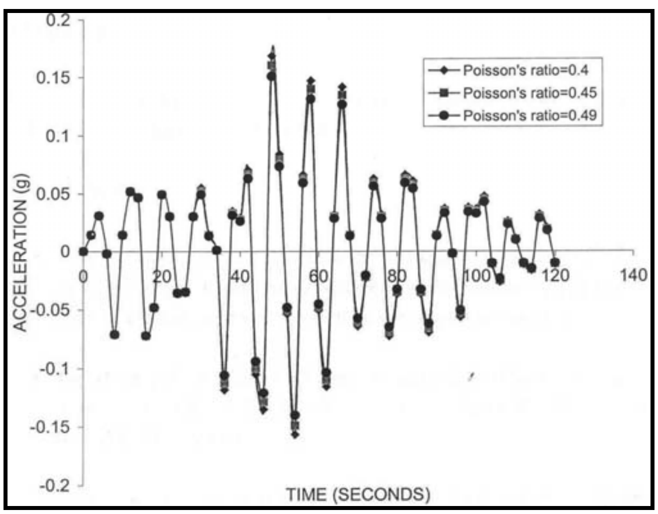


Figure 15: Effect of Poisson's ratio of the core on crest acceleration

CONCLUSIONS

Based on the above study, the following general conclusions are enlisted as follows:

- Application of the MSC.Nastran (Windows) package to a calibration problem showed that the obtained results are in excellent agreement with the reported values. Thus, the program can be used with confidence in analyzing such problems.

- For the problem considered, the computed accelerations, displacements and velocities are observed to be maximal at the crest of the dam. This is consistent with the expected results.

- Poisson's ratio of the core material does not affect the acceleration at the crest significantly in the initial period of vibration. The maximum effect is seen at about 55 seconds, when the Poisson's ratio increases to 0.49 from 0.4. At this instance, the acceleration reduces from 0.17g to 0.15g, the percentage reduction being 11.5%.

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