SEISMIC SAFETY ANALYSIS OF A HIGH ROCK-FILL DAM SUBJECTED TO SEVERE EARTHQUAKE MOTION

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SUMMARY

The seismic safety of 260 m high rock-fill dam in narrow and curved valley, located in seismically active area subjected to Maximum Credeble Earthquake (MCE) with Peak Ground Acceleration (PGA) 0.5g has been investigated. A sequential nonlinear static and nonlinear earthquake analysis has been carriedout using the Mohar-Coulomb material model for the rock-fill material. The high rock-fill dam is found to resist the most severe earthquake ground motion expected at the site. However, the dam undergoes some plastic deformations mostly at the u/s and d/s slopes which is not likely to lead any possible failure. The maximum settlement, peak accelerations and deformations are within acceptable limits under the postulated MCE and thus the dam is found to be seismically safe.

INTRODUCTION

A composite section of a 260m high rock-fill dam consisting of shell, rip-rap, filter and core materials is located in seismically active area and seismic safety of the dam is of prime consideration. Simple stability and linear stress analysis are not adequate for such high dams for severe earthquake motions, which are likely to undergo large plastic deformations. The site dependent response spectra compatible acceleration time history of ground motion has been generated corresponding to 0.5g PGA. Non-linear static and earthquake analyses are carried out to evaluate safety of the dam. Since the dam is in narrow valley and curved in plan, therefore a full cross section for plane strain analysis is not available. Therefore the dam has been analysed at two sections, (i) section which has the maximum core height and (ii) section having maximum downstream slope and short upstream slope. During last couple of decades, there have been major advances in our understanding strong earthquake ground motions, our ability to predict dam response, and in the methods of analysing the safety of dams against earthquakes. Some of these advances are employed to analyse the dam. The non-linear static, the free vibration and non-linear earthquake response analyses of the dam are investigated by finite element method taking into account in variation of material properties with confining pressure.

FAILURE OF ROCK-FILL DAMS

As such, the rock-fill dams are highly resistant to seismic loads due to their large flexibility and capacity to absorb large seismic energy but they are confused with earth-fill dams, which are known to be more prone to earthquake damage. Many high rock-fill dams have been constructed in high seismic areas and many of them have withstood large earthquakes without suffering significant damages. The world's highest rock-fill dams, Nurek dam (300 m) and Rogun dam (335m) are situated in high seismic region. The 146 m high El-Infiernillo rock-fill dam has experienced repeated earthquakes including the famous 1985 Maxico earthquake without suffering significant damage. The 131 m high Miboro rock-fill dam in Japan experienced a 7 Magnitude earthquake with its epicentre only 10 km away from the dam site. The 232 m high Oroville dam located in

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California experienced a 5.8 Magnitude earthquake in 1975 and suffered no damage. Several other cases of rock-fill dams subjected to earthquakes are known, but none of these has undergone substantial damage.

Failure due to excessive settlement and movement of dam are possible modes of failure during earthquake. This type of failure can be expected when the dam rests on weak foundation. Earthquake may result in formation of cracks in the dam leading to leakage. This may be due to differential settlement, movement of faults or severe shaking.

SEISMIC THREAT

The uncertainties associated with the seismic parameters, the occurrence of most severe earthquake during the lifetime of a dam and the prediction of intensity of earthquake shaking at a dam site from the earthquake parameters affect the seismic safety evaluation. Methods of estimating potential earthquake parameters for a dam site are specified in various code of practice[ICOLD]. A high rock-fill dam is designed for Design Basis Earthquake (DBE) and the dam is checked for Most Credible earthquake (MCE).

Input Earthquake Ground Motion: The response spectra compatible earthquake time history has been taken which correspond to Peak Ground Acceleration (PGA) of 0.5g under MCE condition. MCE is the most severe earthquake that may occur once in the lifetime of the dam. Under its effect the dam may undergo distress in the form of cracks (which could be repaired) without undergoing failure. The motion at the base of the dam corresponding to MCE is taken as the input motion for the seismic safety analysis (see Fig.1). The vertical ground motion is taken as 2/3 the strength and in phase with the motion in horizontal direction.

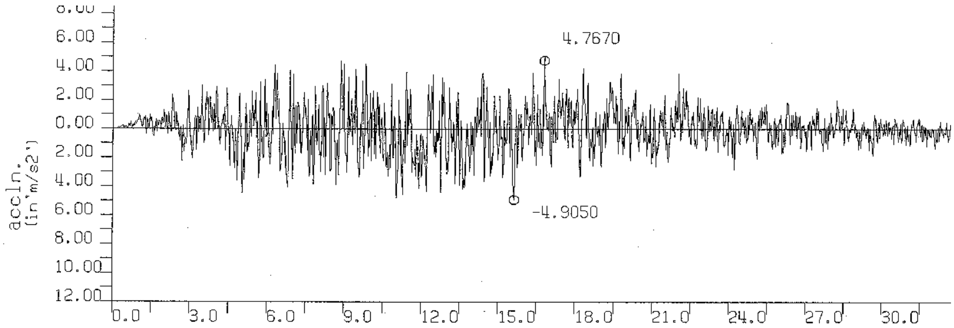


Fig.1 Earthquake Motion at the Dam Site Corresponding to MCE Condition

SEISMIC SAFETY

Rapid advancement in rock-fill dam technology has taken place during the last 5 or 6 decades. Now, technically, there are no restrictions on the size and height of these dams; and consequently, higher and higher dams such as Oroville (235m, USA), Mica (244m, Canada), Chicoasen (264m, Mexico) and Nurek (300m, Uzbekistan) have been constructed in highly seismic regions. Tehri dam (260m, India) and Rogun dam (335m, USSR) are under construction. These have resulted in higher safety considerations in design and construction.

For seismically safe design, the dam has to be protected against the excessive settlement, cracks and stability of slopes. Sufficient defensive measures in design and construction should be taken by ensuring good quality control, adequate compaction of materials, foundation and abutment integrity, ample freeboard, provision of gentle u/s and d/s slope. The defence against the cracks can be realised by introducing a full height upstream filter consisting of cohesionless material, a horizontal drain under the down stream slope and a wide transition/filter zone using a sand/gravel mixture. Defence can also be realised by providing a medium or fine sand zone adjacent to the core containing an appreciable proportion of gravel size particles, but the zone located upstream of the core should not contain too great a portion of coarse particles.

Special care must be given to the specification for construction of the upper part of the dam and at the abutment contacts. The resistant of the upper part of the dam to earthquakes can be increased either by the provision of reinforcement or widening the crest of the dam. The core should be designed to provide resistance to concentrated leakage. Good material is a well-graded mixture of sand, gravel and fines or highly plastic tough clay should be used.

The response of a high rock-fill dam is generally low because the response is much less at the time period of its vibration and therefore these types of dams are suitable from seismic safety considerations. High rock-fill dam may slump/deform under earthquake excitation but it is not likely to break. Considerable free board is provided and slope stabilisation measures are taken. Not a single rock-fill dam has failed due to earthquake and therefore establishes its higher seismic safety.

Concern to dam break situation under earthquake condition has to be addressed from seismic safety consideration. The location of breach, size of breach, rate of release of water and rate of growth of breach will determine whether the release of reservoir water will be sudden or slow. Release of uncontrolled water leading to a dam break situation under the severe seismic threat is very unlikely for such dams. In India, so far dam break analysis under earthquake condition is not a practice and it is not carried out.

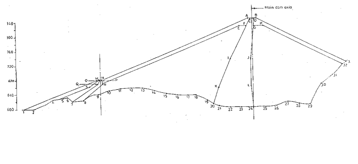
SECTIONS OF THE DAM

A 260.0 m high rock fill dam is located in a narrow valley and curved in plan, the length of the dam at the crest being 570.0 m. The width at the crest is 20.0 m flared to 25.0 m at abutments. The dam section is composed of central impervious core, transition zone, pervious shells and riprap. The dam has an upstream slope of 2.5:1 and down stream slope of 2:1. The core of the dam is moderately inclined and curved on the upstream side. It is a thin core of 0.4H thickness and comprises of gravel sand clay mixture obtained by blending.

Since the valley is narrow and curved at the dam site and therefore a full cross section for plane strain analysis is not available. Two sections corresponding to (i) Section-1 which has the maximum core height and (ii) Section-2 having maximum downstream slope but short upstream slope.

MATHMATICAL MODEL AND DISCRETISATION OF THE SYSTEM

Finite Element Discretization: The 2D sections of the dam and zones identifying the different material is shown in Fig.2. Plain strain idealisation mapped with eight nodded isoparametric elements is used for the analysis. The discretization has been done to map the material boundaries. The nodes at the base of the dam are assumed to be fixed. The water pressure at full reservoir condition is taken as edge load on the u/s portion of the clay core. The discretization of the dam sections is shown in Fig.3. Same finite element mesh is used for multi lift sequential analysis. The Mohr-Coulomb yield criteria is used to model the non-linear behaviour of the rock-fill. The mesh is discretised in such a way that year wise construction layers are easily taken care in the analysis. The numbering of the elements is done horizontally to facilitate the computation for sequential construction.



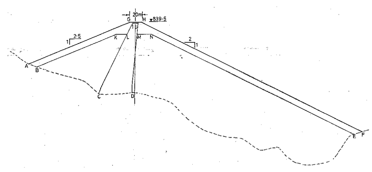
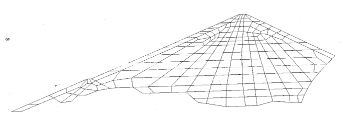


Fig. 2(a) Dam Section -1 **Fig.2(b) Dam Section –2**



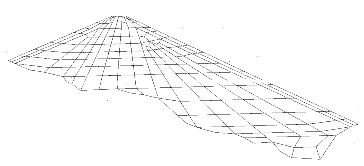


Fig.3(a) Finite Element Mesh - Section 1 **Fig3(b) Finite Element Mesh – Section 2**

ROCK-FILL PROPERTIES

Having determined shear modulus at a confining pressure and shear strain level from tests on rock-fill material, it can be determined at any other confining pressure and shear strain level. Consideration of this effect, the stiffness of dam increases with depth. To take into this variation in the dam analysis, a power law variation of shear modulus has been used. The material shear modulus is assumed to vary with the confining pressure according to following power law.

$$G = G_o(\sigma/\sigma_o)^m \qquad (1)$$

where G is the shear modulus at a confining pressure σ and G_o is the shear modulus at a specified confining pressure σ_o usually $1 \; kg/cm^2$.

The material properties taken for the rock-fill dam analysis are listed in Table 1. This corresponds to a variation of very small shear wave velocity at the top of dam to 437.9 m/s at a depth of 250.0 m in shell material. An average value of shear wave velocity in shell works out to about 315.0 m/s. For core material, these values at a depth of 250m works out to 350.0m/s with an average value of 250.0m/s.

Table-1 Material Properties of Rock-Fill

sl. no.	Property	shell	rip-rap	core
1.	shear modulus, G (t/m^2)	6000.0	6000.0	3000.0
2.	modulus of elasticity E at $10\,t/m^2$ confining pressure	0.162e+05	0.162e+05	5400
3.	Poisson's ratio	0.35	0.25	0.30
4.	moist density (t/m^3)	2.45	2.00	2.00
5.	saturated density(t/m^3)	2.49	2.16	2.15
6.	dry density (t/m^3)	2.36	1.80	1.85
7.	cohesion, c (t/m^2)	7.00	0.50	5.00
8.	effective angle of internal friction (degree)	42.0	42.0	28.0
9.	power variation of G, m	0.5	0.5	0.3

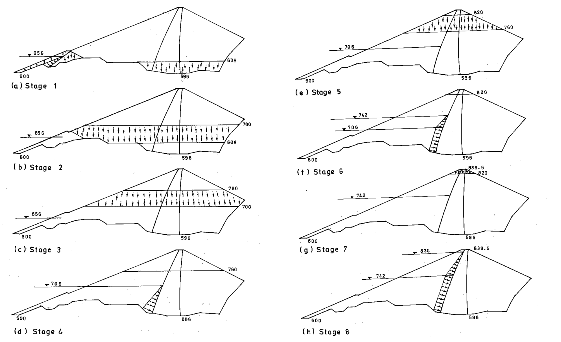
In static analysis, the submerged unit weight has been taken for the u/s shell elements, except the top elements above the phreatic line of u/s shell for which saturated density has been taken. Saturated unit weight for the core and moist density for the d/s shell have been taken. For dynamic analysis saturated unit weight has been taken for the u/s shell elements.

ANALYSIS OF THE DAM

During last couple of decades, there have been major advances in our understanding in our ability to predict dam response, and in the methods of analysing the safety of dams against earthquakes

Static and earthquake response analysis are carried out for both the sections of the dam to study the stability of dam. A 2D-plane strain non-linear multi lift static finite element analysis is carried out to model the construction sequence and the reservoir filling. For the rock-fill and clay core materials, the Mohr Coulomb elasto-plastic material model considering the variation of material properties with the confining pressure has been taken into account [Owen and Hinton(1980)]. The material properties corresponding to the final stress state obtained from non-linear multi-lift analysis have been used for evaluating the free vibration characteristics of the dam. The 2D non-linear elasto-plastic earthquake analysis of the dam for the postulated MCE condition has been carried out and the maximum dynamic displacement and acceleration at dam crest have been worked out. The permanent crest settlement is also worked out at the end of the earthquake excitation.

Static analysis: Analysis of the high rock-fill dam is carried out for static loads due to self weight, water pressure acting on the upstream face of the core and uplift. Analysis is made to determine the internal deformation of the core and the shell, obtaining stress distribution and load transfer within the dam section, location of zones of potential cracking resulting from tensile stresses and investigating the likelihood of hydraulic fracturing. Static deformations of dam are of interest because excessive deformations can lead to loss of free board and danger of over topping. Excessive spreading may lead to longitudinal cracking and adversely affect stability. Differential settlement along the axis of the dam may lead to transverse cracking, which could allow passage of water. Differential settlement between the core and shell can lead to stress reduction in the core and may result in hydraulic fracture.



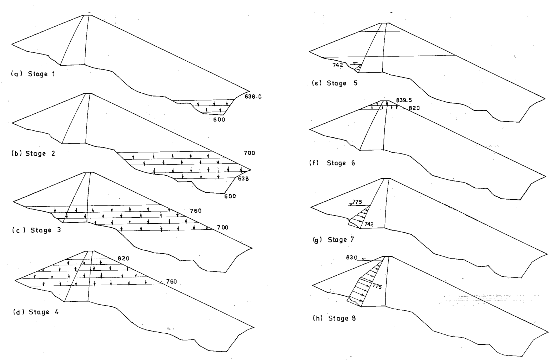


Fig.4(a) Loading sequence in eight stages- Section 1 **Fig.4(b) Loading sequence in eight stages- Section 2**

Multi lift Non-linear Analysis: In the multi lift non-linear static analysis, the simulation of actual sequence of events involved in construction and loading of the dam is possible. The geometry of the finite element mesh can be changed during each layer increment to simulate addition of next layer of fill to the embankment, and water loads can be added in stages, simulating the rise of the water level in the reservoir. They permit simulation of non-linear and stress-dependent behaviour of the rock fill. The values of modulus and Poisson's ratio assigned to each element can be adjusted during each increment of the analysis in accordance with the values of stress calculated in the analysis.

For multi lift non-linear analysis the construction schedule of raising of the dam and the schedule of reservoir filling was taken into account. Figures 4 show the eight stages of construction and reservoir fillings for the two sections. The dam is proposed to be constructed in five years. The figures also show the loading in eight stages. The initial stresses of the previous layer were taken into account in the analysis of the subsequent layers. In the case of impounded water in the reservoir, the weight of the rock-fill under water is taken as submerged while the water pressure is assumed to act on the sloping face of the core as an external load. The analysis is first carried out for the first layer using trail values of E and ν for each element. The stiffness of the dam section is evaluated considering the variation of the Young's modulus with the depth of the dam. The variation of the Young's modulus is computed from the power variation. Since the values of the Young's modulus are known at a confining pressure of 10 t/m2, Young's modulus of elasticity can be computed at any other confining pressure. The power (m) is taken as 0.5 for the shell material whereas for clay it is taken as 0.3. Non-linear variation of the Young's modulus E is computed from the initial stresses obtained by linear static analysis. The evaluated stresses are used to calculate the confining pressure, which is used to calculate the corrected value of E. The procedure is iterative and iterations are continued till convergence is met.

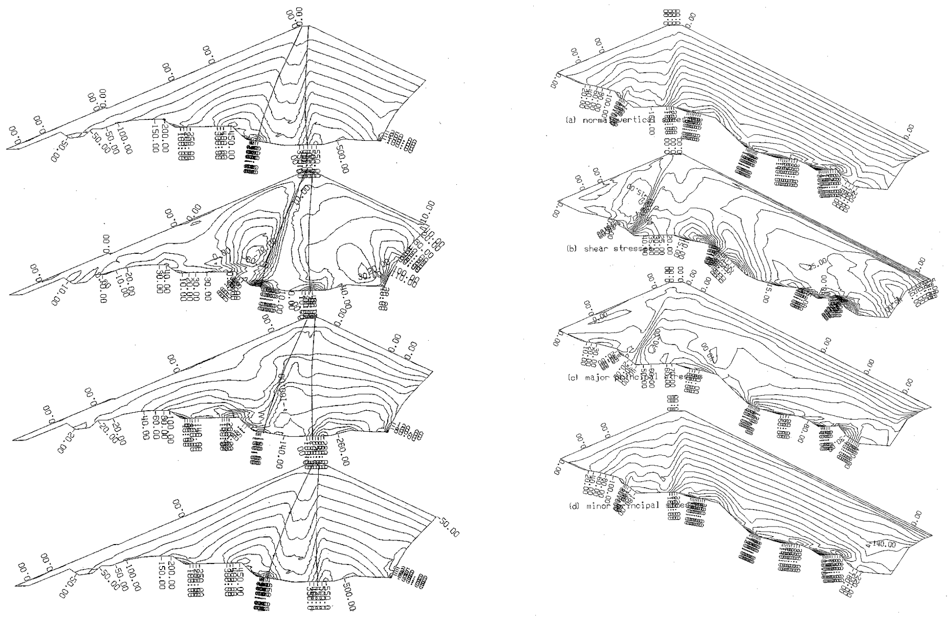


Fig.5 Vertical, Shear, Major and Minor Stress State at the End of Construction for Sections 1 and 2

Figure 5 shows the normal vertical stresses, shear stresses, major principal stresses, minor principal stresses for the two sections at the end of construction and reservoir filling. The analysis shows no tension region in the shell, which is more realistic. Figure 6 shows the contours of horizontal and vertical deformations for the two sections at the end of construction. For Section-1, the maximum horizontal and vertical displacements are found to be 2.0 and 0.3m near the centre of the dam (see Fig.6) and for Section-2 these are found to be 0.6m and 0.4m near the centre of d/s shell (see Fig.7). . It is observed that the plastic strains are confined mostly near the surface of u/s and d/s slopes.

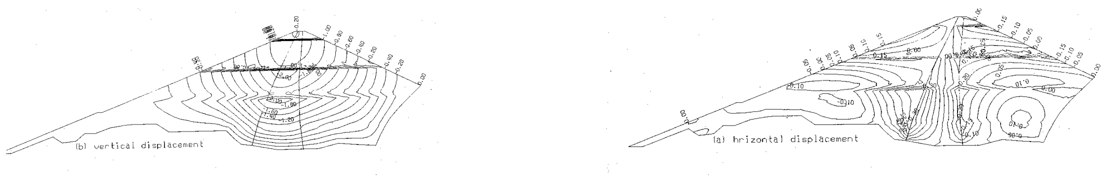


Fig.6 Contours of Horizontal and Vertical Strain at the End of Construction for Section 1

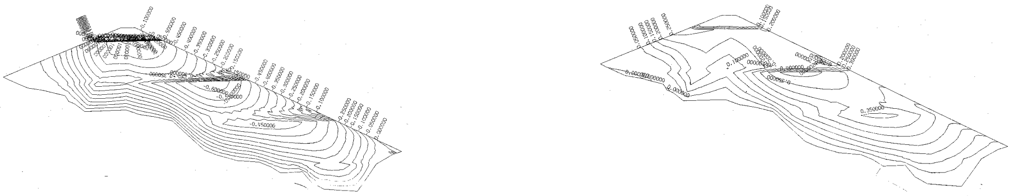


Fig.7 Contours of Horizontal and Vertical Strain at the End of Construction for Section 2

Free Vibration Characteristics: The frequency analysis has also been carried out to study the free vibration characteristics of the dam based on the final stress distribution obtained from multi-lift non-linear static analysis. The time periods of vibration of the dam for the two sections for the first eight modes are given in Table-2. The fundamental vibration mode is a lateral translation mode and the second mode is primarily a vertical translation mode. The translations are much more pronounced near the crest of the dam.

The two sections of dam vibrate in different natural frequencies but in actual behaviour all sections of dam will vibrate in the same mode. In fact the fundamental time period of the dam will be shorter than that obtained by 2D section analysis. This is because the restrain provided the narrow caynon [Gazetas and Dakoulas(1992)].

Table-2 Time Periods of Vibration for the Two Sections

	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8
Section-1	2.2427	1.5640	1.3744	1.3042	1.0576	1.0203	0.9086	0.8273
Section-2	1.6276	1.0923	1.0027	0.8768	0.8250	0.7439	0.7236	0.6918

Earthquake response analysis: Since both the sections are part of the same dam and should be vibrating with the same frequencies. The earthquake response is evaluated considering the same fundamental frequency for the two sections. This is achieved by modifying the material properties in earthquake analysis. For non-linear earthquake analysis, the multi-lift non-linear static analysis at the end of construction is used to get the initial stresses in the dam body. Evaluation of the rock-fill dam response to the base excitation is then carried out using appropriate rock-fill model and rock-fill properties. The rock-fill and the clay core materials have been modelled by Mohr Coulomb elasto-plastic material model considering the variation of material properties with the confining pressure. The damping in the first two modes of vibration is taken as 10%. Response of the dam include the evaluation of stresses, deformations and stability of cross-section of the rock-fill dam. Figure 8 shows the time history of the horizontal/ vertical accelerations and displacements at the dam crest for Section-1 and Fig.9 shows same values for Section-2. Table-3 gives the maximum horizontal/ vertical accelerations and displacements at the dam crest. Figure 10 shows the contours of plastic strain at the end of earthquake. It is observed that the plastic strains occur mainly near the u/s and d/s slope of the dam.

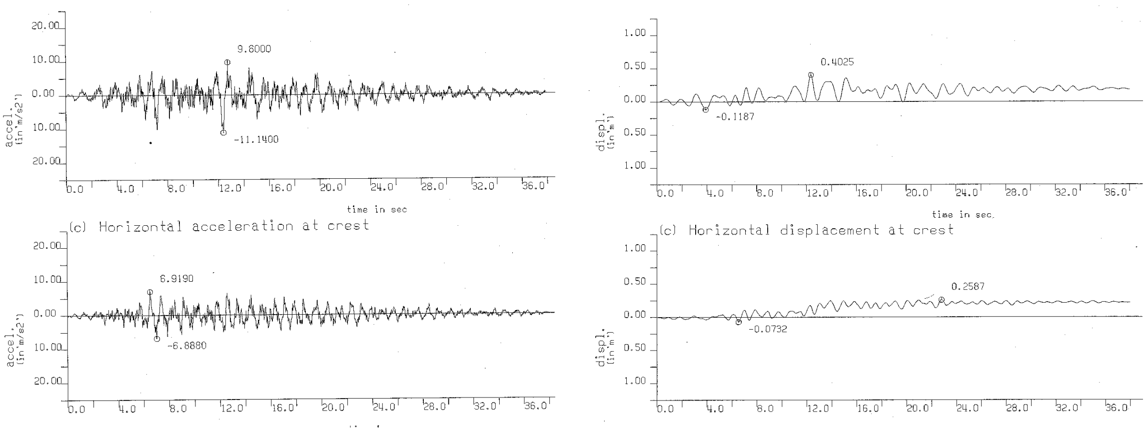


Fig.8 Transverse Vertical Accelerations/ Displacements at Dam Crest for Section-1

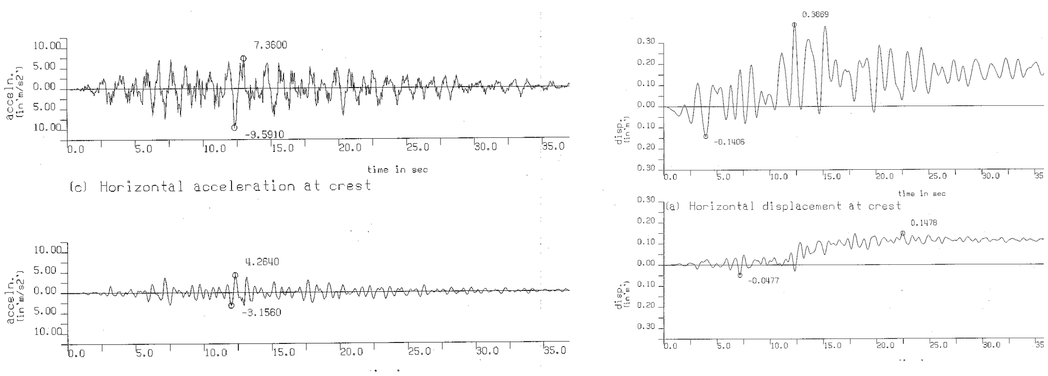


Fig.9 Tranverse and Vertical Accelerations/ Displacements at Dam Crest for Section-2

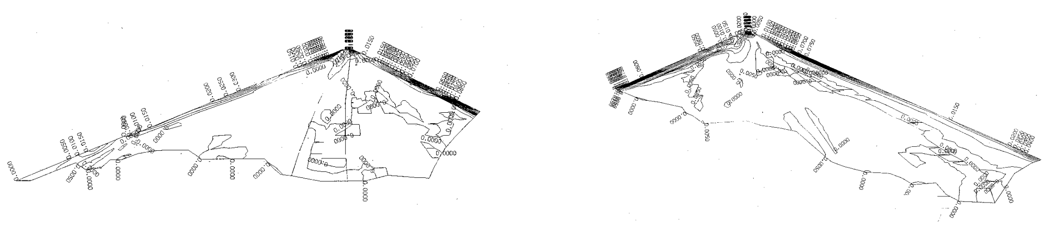


Fig.10 Contours of Plastic Strain at the End of Earthquake for the Two Sections

Table-3 Summary of Maximum Earthquake Response Under MCE Condition

Response parameters	Section-1	Section-2
Maximum horizontal acceleration at dam crest	1.136g	0.978g
Maximum vertical acceleration at dam crest	0.703g	0.434g
Maximum horizontal displacement at dam crest (cm)	40.250	38.690
Maximum vertical displacement at dam crest (cm)	25.870	14.780
Maximum plastic strain	0.080	0.080

CONCLUSIONS

The sequential non-linear static analysis, the free vibration analysis and the earthquake response analysis of the rock-fill dam has been carried out taking into account the effect of material property variation with confining pressure, non linearity, the schedule of raising the dam and the schedule of reservoir filling. The non-linear sequential and earthquake response analyses have given the insight of the dam response with respect to stresses, plastic deformations, peak accelerations and displacements. The high rock-fill dam is found to resist the most severe earthquake ground motion expected at the site. However, the dam undergoes some plastic deformations mostly at the u/s and d/s slopes which may not lead to any possible failure. The plastic deformations in the rock-fill material means adjustment of rock-fill material by undergoing plastic deformations. The maximum settlement, peak accelerations and deformations are found to be within acceptable limits under the postulated MCE at the site and thus the dam is found to be seismically safe.

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