Estimation of Permanent Deformations of a High Dam Due to Earthquakes

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Abstract

The objective of this paper is to compare earthquake induced permanent deformation of a large dam, like Tehri, predicted by finite element method and various existing empirical and semi-empirical methods. A two-dimensional finite element analysis (PLAXIS2D) and five different semi-empirical and empirical methods, like, Seed and Makdisi's method, Newmark's double integration method, Jansen's method, Swaisgood's method and Bureau's method have been utilized in this study to obtain the probable dynamic behavior of the dam and their results compared to get a range of values within which, the permanent deformation of the dam, is estimated to lie. The maximum deformation (755 cm for M=8.5 and 43 cm for M=7.0) are predicted by Seed and Makdisi's method while the minimum deformations (14 cm for M=8.5 and 2.5 cm for M=7.0) are computed by Jansen's method.

Introduction

Estimating the permanent deformations that an embankment dam will undergo during an earthquake shaking is a very difficult task. The effort is made more difficult by the myriad of factors that are involved and lack of reliable field data. The ground vibrations at a site are unique to the particular earthquake causing them and to the site-specific conditions existing at the dam. The characteristics of a dam, such as, type of construction, structural height, upstream and downstream reservoir levels affect the response of the structure. The sophisticated analytical tools available today, like finite element method is the most recommended method. However, realistic deformation values may be expected from finite element analyses only when the material models and the material parameters are capable of accurately simulating real life scenario. Though significant progress has been made in modeling material behavior still most of the material models perform poorly when comes to reality check. Even with these limitations, dynamic finite element analysis of embankment dam is recommended for proper evaluation of its seismic safety.

Over the years, starting with the fifth Rankine lecture by Newmark [1], several simplified methods have been proposed to estimate permanent deformations of embankment dams due to earthquakes. Most of these methods are empirical or semi-empirical in nature and based on statistical analyses of data from a limited number of case histories. This paper reviews some of these simplified methods and compares their performance with that of a finite element method in estimating permanent deformation of a large dam, like Tehri Dam, subjected to two hypothetical earthquakes, one with magnitude (M_w) 7.0 and peak ground acceleration (PGA) of 0.23g and another with magnitude (M_w) 8.5 and PGA of 0.45g.

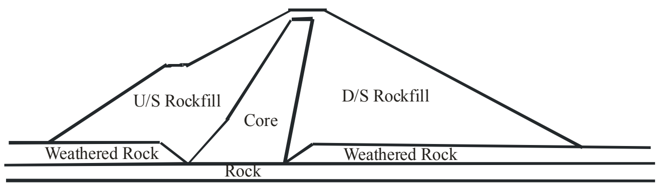

Figure 1: Typical Cross-Section of Tehri Dam.

Tehri Dam

The Tehri Dam is located very near the town of Tehri in the Garhwal region of Uttaranchal in India. The rockfill dam is built on the Bhagirathi River. The dam is planned to be the fifth highest dam in the world. It is 260.5 m in height at the deepest point. The crest is 20 m wide and spans 574 m across the valley. The upstream slope of the dam is 2.5(H): 1(V). The downstream slope is 2(H): 1(V). The rockfill dam has an inclined impervious core made of clayey materials and upstream-downstream shells of graded gravel topped with blasted rocks. A detail description of the dam is given by Thatte [2]. Figure 1 shows a typical cross-section of the dam. Table 1 shows the drained strengths of the dam materials. The objective of this paper is to investigate the seismic behavior of a large dam, like Tehri Dam, when subjected to an M_w=7, PGA=0.23g earthquake for which the dam has been designed and an M_w=8.5, PGA=0.45g hypothetical earthquake which some (Gaur [3]) believe to be more probable earthquake in the Himalayas.

TABLE 1: MATERIAL STRENGTH PARAMETERS.

Zones of dam	Densities in t/m^3		Cohesion c'	Friction Angle, ϕ'
	Moist	Saturated		
U/S Rock fill	1.92	2.16	0.00	40.00
D/S Rock fill	2.08	2.24	0.00	35.00
Core	1.86	2.00	0.00	30.00
Rock, & Weather Rock		2.30	0.00	45.00

Simplified Methods for Determining Permanent Deformations

Seed and Makdisi's Method

The simplified procedure developed by Makdisi and Seed [4] follows the premise that permanent deformation takes place whenever the rigid body acceleration, K_{max}, of a potential sliding mass exceeds the yield acceleration, K_y, for that mass. The yield acceleration is determined by performing a series of pseudo-static analyses. Makdisi and Seed have related the rigid body acceleration for various sliding masses to the peak acceleration at the crest of the dam and to the depth of the sliding mass. The yield acceleration, K_y is defined as that average acceleration which produces a horizontal inertia force on a potential sliding mass to yield a factor of safety of unity and thus causing it to experience permanent displacements. Yield accelerations are determined for three potential sliding masses on the upstream slope and three sliding masses on the downstream slope of Tehri dam. The locations of the slide surfaces are shown in Figure 2. The upstream and downstream water levels are assumed at 830m (maximum normal operating pool) and 594m, respectively during an earthquake. The stability analyses are performed according to Simplified Bishop's method. Table 2 summarizes the results of the static stability analysis and the values of yield acceleration, K_y for all the six cases.

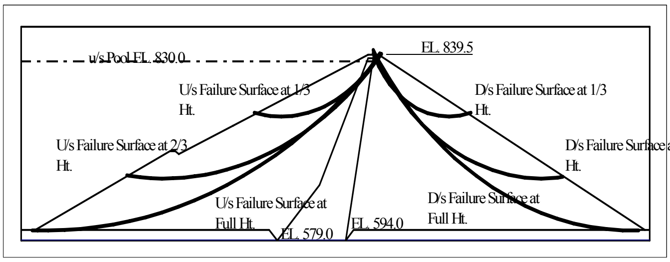

Figure 2: Location of U/s and D/s Failure Surfaces.

The maximum crest acceleration, u_{max}, and the natural period of the dam, T_o are obtained by response spectrum analysis of the dam modeled as a shear beam of variable stiffness (triangular shape). The shear beam procedure is adapted for soils by following equivalent linear approach that amounts to calculating dynamic soil properties iteratively, until those properties are compatible with the calculated strain level. The assumed response spectra at 5%, 7%, 10% and 15% of critical damping for the M_w=7, PGA=0.23g earthquake are shown in Figure 3.

TABLE 2: YIELD ACCELERATIONS.

Zones of dam	Location of Surface	Factor of safety (FS) in static case	Yield Acceleration (k_y) (FS of 1)
Downstream	1/3 height	1.61	0.225
Slope	2/3 height	1.51	0.191
	Full	1.49	0.182
Upstream	height	2.37	0.281
Slope	1/3 height	2.47	0.255
	2/3 height	2.35	0.228
	Full height		

The response spectra for the M_w=8.5, PGA=0.45g earthquake are obtained by arithmetic scaling the spectral acceleration for the 0.45g PGA. Convergence to strain-compatible properties of the constitutive dam materials is achieved within two and three cycles of iterations for the 7.0 and 8.5 magnitude earthquakes, respectively. The results of the response spectrum analysis are shown in Table 3.

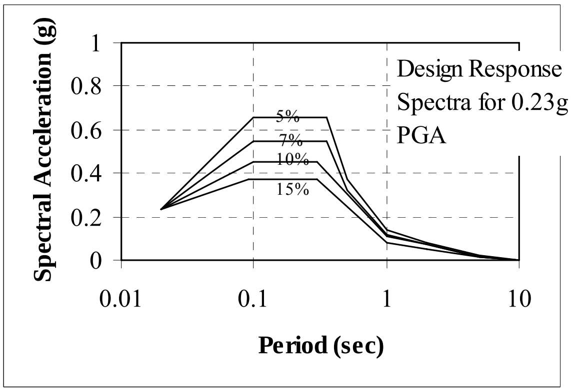

Figure 3: Design Response Spectra at 5, 7, 10 and 15% of Critical Damping.

Makdisi and Seed showed that a unique relationship exists between the yield acceleration, K_y, the depth of slip surface, y/h, and the ratio of the maximum rigid body acceleration, K_{max} to the maximum crest acceleration, a_{max}. After estimating the maximum crest acceleration, a_{max}, the relationship shown in Figure 4 is used to determine the values of, K_{max}, for each sliding mass under the design motion. The upper bound curve is utilized in the present analysis. The values of K_{max} and K_y/K_{max} for all the cases are shown in Table 4. The horizontal displacement, U, for each of the sliding masses is then estimated from the curves shown in Figure 5. These curves (adopted from [4]) relate displacement, U, with the magnitude (M) of earthquake, K_y/ K_{max}, and the period of the dam, T_o.

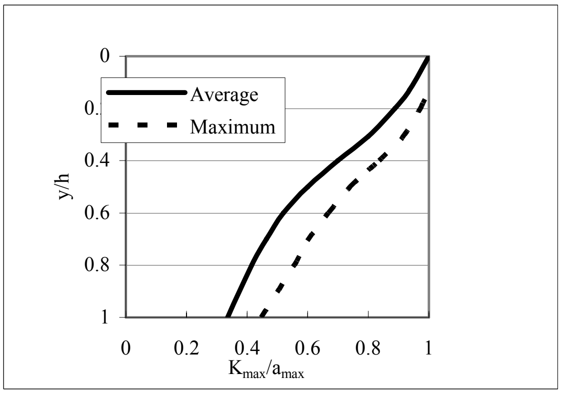

Figure 4: Relationship between y/h and K_{max}/a_{max} [4].

TABLE 3: RESULTS OF RESPONSE SPECTRUM ANALYSIS.

	Magnitude of Earthquake(M)	
	7.0	8.5
Maximum Crest Acceleration, a_{max}	0.62g	0.96g
Predominant Period, T_o	1.10 sec	1.10 sec
Average Equivalent Strain	0.015 %	0.03 %
Strain Compatible Damping	8 %	9 %

TABLE 4: RIGID BODY MAXIMUM ACCELERATION.

Location of Sliding Surface	Depth y/H	Upstream		Downstream	
		K_{max} (g)	K_y/K_{max}	K_{max} (g)	K_y/K_{max}
M = 7.0					
1/3 Height	0.33	0.57	0.50	0.57	0.40
2/3 Height	0.66	0.41	0.62	0.41	0.47
Full Height	1.00	0.29	0.78	0.29	0.62
M = 8.5					
1/3 Height	0.33	0.88	0.32	0.88	0.26
2/3 Height	0.66	0.64	0.40	0.64	0.30
Full Height	1.00	0.45	0.50	0.45	0.40

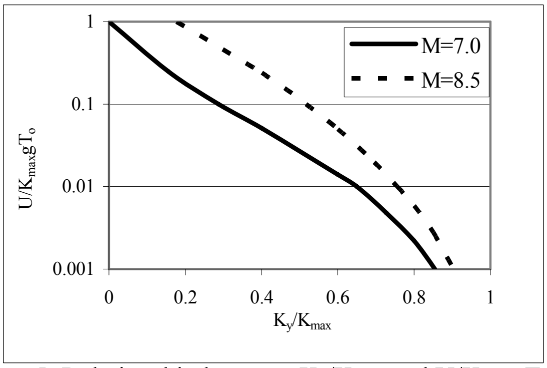

Figure 5: Relationship between K_y/K_{max} and $U/K_{max}gT_o$ [4].

TABLE 5: PERMANENT DISPLACEMENTS BY MAKDISI & SEED'S PROCEDURE.

	Magnitude of Earthquake=7.0					
	Upstream			Downstream		
Surface Location	K_y/K_{max}	$U/[K_{max}.g.T_o]$	U (cm)	K_y/K_{max}	$U/[K_{max}.g.T_o]$	U (cm)
1/3 Ht	0.49	0.045	27.4	0.39	0.07	42.6
2/3 Ht	0.62	0.015	6.6	0.46	0.06	26.5
Full Ht	0.78	0.005	1.57	0.62	0.01	4.73
	Magnitude of Earthquake=8.5					
	Upstream			Downstream		
Surface Location	K_y/K_{max}	$U/[K_{max}.g.T_o]$	U (cm)	K_y/K_{max}	$U/[K_{max}.g.T_o]$	U (cm)
1/3 Ht	0.32	0.600	566	0.25	0.80	755
2/3 Ht	0.40	0.400	275	0.30	0.65	445
Full Ht	0.50	0.100	48.8	0.40	0.35	170

The displacements of the Tehri Dam thus obtained for M=7.0 and M=8.5 magnitude earthquakes are shown in Table 5.

Double Integration Method

This method of computing the earthquake-induced deformation is based on the sliding wedge analogy proposed by Newmark [1]. According to this method, sliding of a failure mass occurs whenever the inertia of the mass exceeds the frictional resistance along the sliding surface. The frictional resistance is characterized by the yield acceleration, K_y. Relative displacements are calculated by double integration of the difference between mass acceleration and yield acceleration.

Following the procedure developed by Seed, et al. [5], the M=8.5 earthquake is modeled by 26 cycles of identical full sinusoidal waves. The average value of acceleration amplitude, a_{avg} is calculated as 2/3 of PGA (PGA= 0.45g) and found to be 0.3g. The 7.0 magnitude earthquake is modeled by 15 cycles of identical full sinusoidal waves. The average value of acceleration amplitude, a_{avg}, for this case is found to be 0.15g. The equivalent time history of acceleration may then be represented by a sine wave given by:

$$K(t) = a_{avg} Sin\left(2.\pi.\frac{t}{T}\right) \qquad (1)$$

The calculated displacements are a function of T, period of the acceleration time history. The period of the motion is a function of the foundation conditions, magnitude of the earthquake, distance to the source and intensity of shaking. For embankment dams it typically lies between 0.1 second and 1.0 second. For the M=7.0 magnitude earthquake, the yield accelerations for the upstream and downstream slopes are greater than the average value of acceleration amplitude, a_{avg} , of 0.15g. Thus the deformations of both the slopes of Tehri Dam are negligible for the M=7.0 magnitude earthquake. However this is not the case for M=8.5 magnitude earthquake. It is assumed that the deformation is triggered at time, t_1, when the acceleration of the base exceeds the yield acceleration, K_y. t_1 and t_2 are the limits within which the ground acceleration (K) exceeds yield acceleration (K_y). The values of the limits t_1 and t_2 are computed for both the cases by solving the following equations:

$$K(t_1) - K_y = 0 \quad (2)$$

$$K(t_2) - K_y = 0 \quad (3)$$

The predominant period of the dam is 1.1 sec from response spectra analysis. The values of t_1 and t_2 at this period of motion are found to be 0.137 sec and 0.363 sec, respectively for the upstream slope and 0.104 sec and 0.396 sec, respectively for the downstream slope of the dam.

t_3 is the time at which mass velocity, V, equals resisting velocity, V_y. The value of t_3 can be obtained by solving the following equation:

$$\int_{t1}^{t3} K(t)\,dt - \int_{t1}^{t3} K_y\,dt = 0 \quad (4)$$

The value of t_3 is 0.482 sec for the upstream slope and 0.558 sec for the downstream slope of the Tehri Dam.

An expression for the resisting velocity, V_y can be obtained by integrating the yield acceleration, K_y:

$$V_y(t) = K_y.t - K_y.t_1 \quad (5)$$

An expression for mass velocity, V, is obtained by integrating Equation 1 as follows:

$$V(t) = -a_{avg}\frac{T}{2.\pi}.Cos\left(\frac{2.\pi.t}{T}\right) + a_{avg}\frac{T}{2.\pi}.Cos\left(\frac{2.\pi.t_1}{T}\right) \quad (6)$$

The relative displacement, d, of the dam at the end of each cycle is then computed by integrating the difference between the mass velocity and the yield velocity as follows:

$$d = \int_{t_1}^{t_3}\left(V(t) - V_y(t)\right)dt \quad (7)$$

Total relative displacement of the Tehri Dam at the end of an M = 8.5 earthquake is estimated to be between 52.8 cm (for the upstream slope) and 147.4 cm (for the downstream slope).

Jansen's Method

Jansen [6] developed the following empirical relationship between earthquake magnitude, M, the maximum crest or near crest acceleration, K_m, the yield acceleration, K_y, and the total settlement at the crest, U:

$$U = [48.26(M/10)^8(K_m - K_y)]/\sqrt{K_y} \quad (8)$$

The value of K_m in the above equation can be obtained from Figure 7.

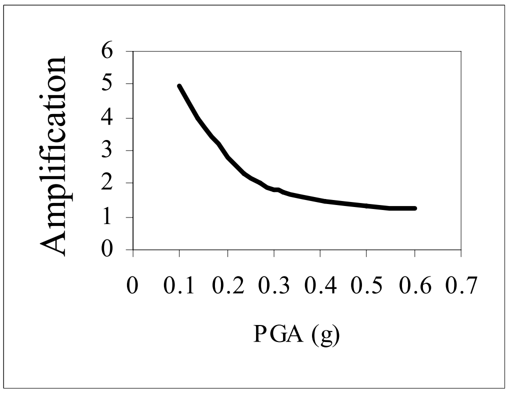

Figure 7: Amplification at Dams during an Earthquake.

The total settlement at the crest, U, at different surface location are shown in the Table 6.

TABLE 6: ESTIMATE OF TOTAL SETTLEMENT OF TEHRI DAM FROM JANSEN'S METHOD.

K_m (in g)	M	Location of Critical Sliding Surface	Upstream Slope		Downstream Slope	
			K_y (g)	U (cms)	K_y (g)	U (cms)
0.5	7	1/3 Height	0.28	1.45	0.22	1.95
		2/3 Height	0.25	1.66	0.19	2.34
		Foundation	0.22	1.93	0.18	2.45
0.6	8.5	1/3 Height	0.28	8.91	0.22	11.51
		2/3 Height	0.25	10.03	0.19	13.50
		Foundation	0.22	11.35	0.18	14.13

The maximum deformation estimated by Jansen's method is between 1.93 cm and 2.45 cm for M = 7 earthquake. While the maximum deformation is estimated to be between 11.35 cm and 14.13 cm for an M = 8.5 earthquake.

Swaisgood's Method

Swaisgood [7] related the crest settlement, Δ (expressed as percentage of the combined dam and alluvium thickness) to a Seismic Energy Factor (SEF), dam type (Ktyp), dam height (H), and depth of alluvium (At) as follows:

$$\Delta(\%) = SEF \times Ktyp \times Kdh \times Kat \quad (9)$$

The seismic energy factor (SEF) in the above equation is dependent on the possible magnitude of earthquake (M) and peak ground acceleration (PGA) at the dam site and is expressed as:

$$SEF = e^{(0.7168*M+6.405*PGA-9.098)} \quad (10)$$

The factor Ktyp depends on the type of dam construction. Ktyp is 1.187 for Earth Core Rockfill and Concrete Faced Rockfill Dams, 1.363 for Earthfill Dams, and 4.620 for Hydraulic Fill Dams. The factor Kat depends on the alluvial thickness (At) present beneath the dam. The greater the depth of alluvium, the greater is the deformation in the dam. It also reflects the fact that the natural periods of vibration with deep and soft soil deposits are longer than that of rock sites with no alluvium.

$$Kat = 0.851 * e^{(0.00368*At)} \quad (11)$$

The factor Kdh relates dam height (H) to the settlement as follows:

$$Kdh = 9.134 * H^{-0.437} \quad (12)$$

The above factor indicates that the higher dams settle less than smaller dams. This may be due to the fact that the resonant frequencies of shorter dams are closer to the natural frequencies of the earthquake vibrations. The following table summarizes all the Swaisgood's factors and the estimated settlements of the Tehri Dam for the two earthquakes.

TABLE 7: ESTIMATE OF CREST SETTLEMENTS AT TEHRI DAM BY SWAISGOOD'S METHOD.

M	SEF	Ktyp	Kdh	Kat	Rel. Settle ment,%	Crest Settle ment,in cm
7.0	0.07	1.19	0.48	0.85	0.034	8.8
8.5	0.9	1.19	0.48	0.85	0.43	112.0

Bureau's Method

Bureau [8] related relative crest settlement (%) to the Earthquake Severity Index (ESI) (Figure 8). The earthquake severity index (ESI) was defined as:

$$ESI = PGA * (M - 4.5)^3 \quad (13)$$

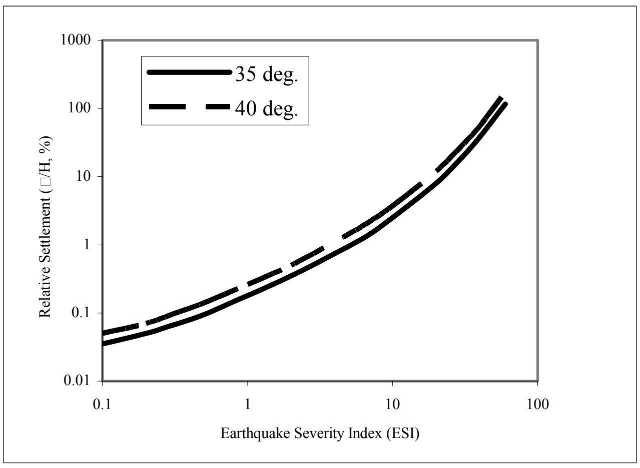

Figure 8: Relative Settlement (%) Vs. ESI [8].

The crest settlements of Tehri dam as obtained by this method are shown below.

TABLE 8: CREST SETTLEMENTS BY BUREAU'S METHOD.

Peak Ground Accl. (PGA)	M	ESI	Crest Settlement (cms)
0.23g	7.0	3.59	18-23.4
0.45g	8.5	28.8	521- 782

The Finite Element Method

In this study, a finite element program called, PLAXIS2D [9] is utilized for the dynamic analyses of the Tehri Dam. The numerical analysis is done in three stages. In the first stage, the gravity force is turned on. In this stage the undrained behavior of soil is ignored. In the next stage, the static analysis is done where the dam is built and the reservoir is impounded. In the third stage, the dynamic analysis is carried out by specifying acceleration time history of the selected earthquake. The output in terms of acceleration, deformation, pore pressures and stresses are viewed at the end of each stage of analysis. In absence of any reliable data, the 23secs of the 1979 Mexico Earthquake (M=7.6) ground motion was selected. The ground motion was scaled to 0.23g and 0.45g, and applied at the base of the dam. Figure 9 shows the selected ground motion scaled to 0.23g. The numerical analyses predicted no liquefaction of the dam and its foundation during the earthquakes. The vertical deformation was computed at the crest of the dam while horizontal deformation was obtained at the upstream berm of the dam. Figure 10 shows the vertical deformations at the crest of the dam for the 0.23g and 0.45g earthquakes. Figure 11 shows the horizontal deformations of the upstream berm of the dam for both the earthquakes.

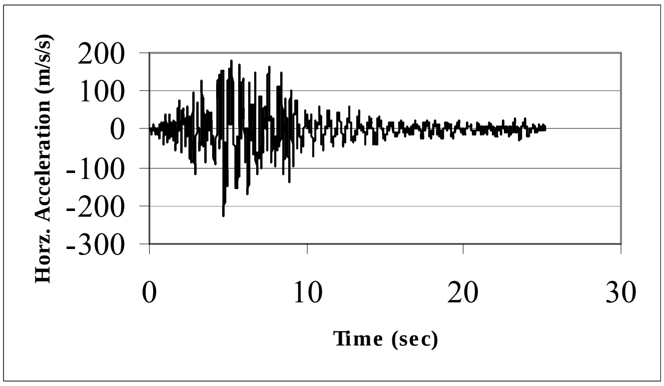

Figure 9: The Selected Ground Motion Scaled to 0.23g.

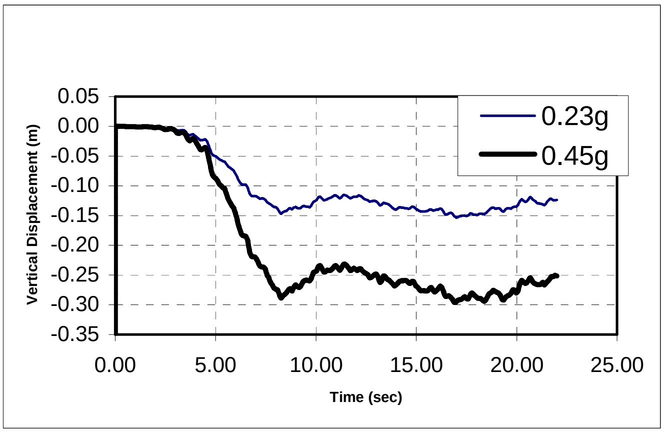

Figure 10: Vertical Deformations at the Crest of the Dam.

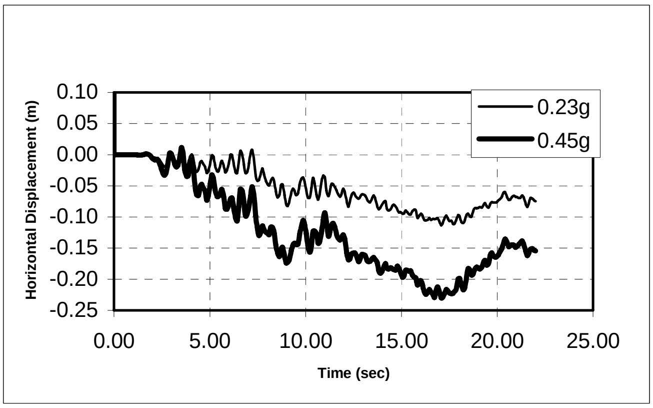

Figure 11: Horizontal Deformations at the U/S Berm.

Results

Table 9 summarizes a comparison of the deformations of Tehri dam predicted by different empirical methods and a finite element method for M=7.0 and M=8.5 earthquakes.

TABLE 9: DEFORMATIONS BY DIFFERENT METHODS.

Method Used	Deformations (cms) at M=7.0		Deformations (cms) at M=8.5	
	U/S	D/S	U/S	D/S
Seed & Makadisi's	27.44	42.68	566.53	755.37
Double Integration	----	----	52.80	147.00
Jansen	1.93	2.45	11.35	14.13
Swaisgood	8.80	8.80	112.00	112.00
Bureau	18.00	23.40	521.00	782.00
Finite Element	50.00	20.00	110.00	51.00

Conclusions

The maximum deformations occur along the upper reaches of the upstream face of the dam while the surface at the foundation level has almost negligible deformations.

For the M=7 earthquake, the maximum deformation is predicted by the finite element method, while for the M=8.5 earthquake, Seed and Makdisi's method predicts the maximum deformations.

All the simplified methods predict larger deformation on the downstream face of the dam, but finite element method predicts almost two times larger deformations for the upstream face of the dam.

Among the simplified methods, the maximum deformations are computed by Seed and Makdisi's method while the minimum deformations are obtained by Jansen's method.

The large variation of the seismic deformations predicted by different methods indicates scope of more work in this area and stresses on the need for the instrumentation of the dams and verification of different methods in predicting seismic deformation of dams.

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