International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

1981 - First International Conference on Recent Advances in Geotechnical Earthquake Engineering & Soil Dynamics

30 Apr 1981, 1:30 pm - 5:30 pm

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Proceedings: First International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 26 - May 3, 1981, St. Louis, Missouri

Permanent Deformation of Earth Dams Under Earthquakes

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SYNOPSIS. Assessment of dam embankment deformation by various methods such as those proposed by Newmark, Ambraseys and Seed was reviewed herewith. A simple method based on dynamic response spectrum analyses by step-by-step integration technique is proposed for independent permanent deformation evaluation. The proposed method assumes that failure occurs on a well defined slip surface and that the material behaves elastically at stress level below failure but develops a perfectly plastic behavior above the yield acceleration. The results of the proposed method were compared with those obtained from other method for actual examples. Favorable agreement on the analytical results was achieved.

INTRODUCTION

Near catastrophic failure of San Fernando dam due to the earthquake in 1971 and the complete failure of Oshima Tailing dam in Izu-Oshima earthquake of 1978 demonstrated vunerability of earth dams under moderate to strong earthquakes. As reported and as shown in Table 1, by Romo et al, (1980), Shen et al, (1980) and Seed et al. (1980) there are about 300 dams suffered damages due to earthquakes since 1906. At least 10% of these dams suffered complete failures. All damages are permanent deformations evidenced by slumping, sometime cracking of embankment and sliding of slopes. Because of catastrophic nature of a dam failure, these permanent deformations must be evaluated and be taken cared of at the early stage of designs of an earth dam.

The ground accelerations during a moderate to strong earthquake can cause large inertia forces throughout the embankment. These induced inertia forces can temporarily exceed the shear strength of the embankment for a short period and result in permanent deformations of the dam. The permanent deformations can not be adequately evaluated by the pseudo-static method. This is demonstrated by the fact that both San Fernando dam and the Oshima tailing dam were analyzed by the pseudo-static method for seismic coefficients of 0.15g and 0.20g respectively. They were found to have safety factors greater than 1.0 prior to their failures. Various methods has been proposed by various engineer and investigators to assess the permanent deformations in an earth dam due to earthquakes. The well known methods are those proposed by Newmark (1965) Makdisi and Seed (1978), Ambraseys (1974) and Lee (1978).

A simple method based on dynamic response spectrum analyses by step-by-step integration technique is proposed for independent permanent deformation evaluation. The proposed method uses the original Newmarks's concept of a sliding block for calculating permanent deformations. It assumes that failure occurs on a well-defined slip surface and the material behaves elastically at stress levels below failure surface but develops a perfectly plastic behavior above yield accelerations. Yield accelerations are assumed to vary during the Seismic motion due to reduction in shear strength of materials in an embankment.

The proposed method was used in actual design examples and the results were compared with those obtained from other methods.

METHODS FOR COMPUTING PERMANENT DEFORMATIONS

1. Newmark Method

Newmark (1965) has shown that the permanent displacement of a sliding mass relative to the base is the sum of increments of displacement occurring during a number of individual pulse of ground motion. Whenever the ground acceleration exceeds the yield acceleration, sliding will occur along the failure plane and the magnitude of the displacement is computed by double integration of the acceleration time history. By assuming the resistance of the sliding to be rigid-plastic and unsymmetrical for an embankment that suffers a slope failure due to seismic ground motions, the average earthquake-induced horizontal displacement is given by:

$$U_m = \frac{V^2}{2gN} (1 - N/A) \qquad (1)$$

where V is the maximum ground velocity, A is the maximum ground accelration and N is the critical acceleration at which a potential sliding surface attains a factor of safety of unity. The relative displacement will be permanent if no further relative motion is occurred. Furthermore, the freeboard loss, L, can be calculated using the following relationship:

$$L = U_m \tan\delta \qquad (2)$$

where δ is the angle of the sliding plane with the horizontal.

The Newmark charts for computing the permanent deformation were made for normalized earthquakes of maximum acceleration of A=0.55g and maximum velocity of 30 in/sec. Indirect computation of permanent displacement for an unsymmetrical sliding can be made by using the following equation:

$$U_m = U_s \frac{V^2}{1800A} \qquad (3)$$

where U_s is the normalized displacement from the Newmark Chart (See Figure 1).

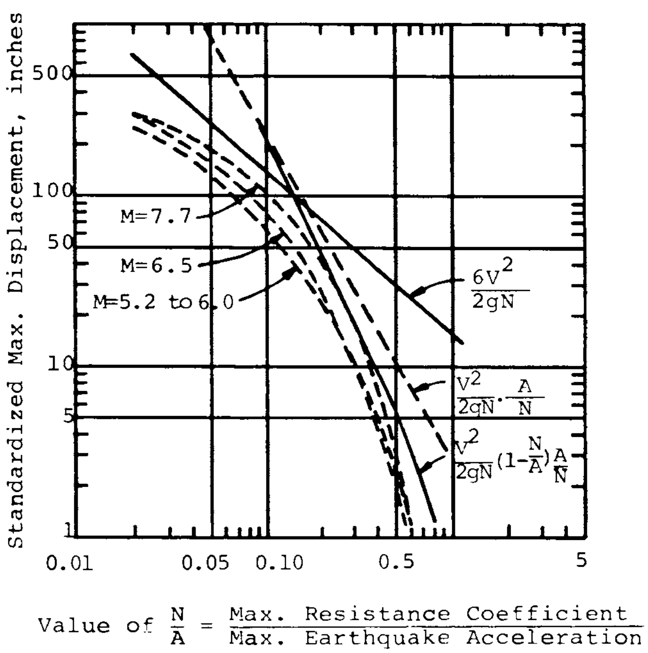

Figure 1. Mean Permanent Displacement for Different Magnitudes of Earthquakes (Soil Sites)

2. Ambraseys Method

With respect to residual displacements in an earth dam, Ambraseys (1974) has developed an upper bound empirical equation for a crude evaluation. The equation is:

$$\log(U) = 2.3 - 3.3\ (Kc/Km) \qquad (4)$$

where U is the residual displacement in centimeter, Kc is the critical acceleration needed to reduce the factor of safety to one, and Km is the maximum input ground acceleration. Equation (4) is useful only for earthquakes magnitude less than 6.5 and $0.1 < Kc/Km < 0.8$ and sloping surfaces from 2:1 to zero.

Ambraseys (1974) also proposed the following equation for computation of the critical acceleration.

$$K_c = \frac{\tan\emptyset - \tan\beta}{1 + \tan\emptyset' \tan\beta} \qquad (5)$$

where $\emptyset'$ is the angle of internal friction of the material, and β is the slope angle of the dam. As can be seen from Equation (5) the critical acceleration K_c is a function of the geometry of the mass as well as the soil properties and static factor of safety of the mass profile.

TABLE 1. Damages to Embankment in Earthquakes, 1900-1980

Earthquake Location	Year	Magnitude	Number of Embankment: Damages	Number of Embankment: Failures
In U.S.A.				
San Francisco	1906	8.25	5 Dams	0
Kern County	1952	7.6	3 Dams	1
Fallon	1954	6.7	2 Dams 2 Dams	1
San Fernando	1971	6.6	2 Dams	0
Santa Barbara	1925	6.3	1 Dam	1
El Centro	1940	6.6	Several dikes	Several dikes
Hebgen Lake	1959	7.6	1 Dam	1
Alaska	1964	8.4	1 Dam	1
In Japan				
Kanto	1923	8.2	3	0
Ojika	1939	6.6	74 Dams	12
Fukui	1948	7.3	1 Dam	1
Kita-Muto	1961	7	0	0
Tokachi-Oki	1968	7.8	93 Dams	9
Izu-Oshima	1978	7	1 Dam	1
In Mexico				
Volcano Lake	1915	N.A.	1 Dike	1
Guerrero	1979	7.6	2 Dams	0
In China				
Bachu	1961	6.8		
Longyao	1966	6.8		
Ningji & Dongwang	1966	7.2		
Bohai Gulf	1969	7.4		
Yangjang	1969	6.4		
Tonghai	1970	7.7	112 Dams	0
Haicheng	1975	7.3		
Longlin	1976	7.3		
Tengshan	1976	7.8		
Songpan	1976	7.2		
Liyang	1979	6.0		
	TOTAL		311	36

3. Makdisi and Seed's Method

The dynamic procedures for computing the deformations of earth dams during earthquakes have been proposed by Makdisi and Seed (1978). The proposed approach which is equivalent to Newmark's approach except that the earthquake excitation is obtained from the dynamic response of the embankment using either shear-beam or finite element models. The method assumes perfectly elastoplastic soil behavior. Values of yield acceleration are a function of the embankment geometry, the undrained strength of the material (or the reduced strength due to shaking), and the location of the potential sliding mass. The numerical application of this method can be carried out using Figures 2 and 3. The yield accelerations are obtained from the pseudo-static slope stability analyses. The basic steps required in the computations are:

a) Determine the yield acceleration from the pseudo-static stability analysis.

b) Determine the maximum acceleration ratio K_{max}/U_{max} from Figures 2 for various depths of the sliding mass. In the ratio, K_{max} is the average maximum acceleration of the sliding mass and U_{max} is the maximum crest acceleration.

c) Evaluate the magnitude of the normalized displacement from Figure 3.

4. Step-by-Step Integration Method

Wilson and Clough (1962) has shown that the equilibrium of a single degree system at time "t" is expressed by the following equation for a viscous form of damping:

$$\ddot{X}_t + 2\lambda\omega\dot{X}_t + \omega^2 X_t = \ddot{U}_g(t) \qquad (6)$$

where

X_t = the relative displacement of the system respect to the foundation.

$\dot{X}_t$ = the velocity of the system.

$\ddot{X}_t$ = the acceleration of the system.

λ = the damping ratio.

ω = the natural frequency in radian per second.

$\ddot{U}_g$ = the ground acceleration.

Assumption of the linear acceleration within a time increment, Δt, leads to a parabolic variation of velocity and a cubic variation of displacement within the time increment and yields the following equation:

$$\ddot{X}_t = F[\ddot{U}_g(t) - 2\lambda\omega a - \omega^2 b] \qquad (7)$$

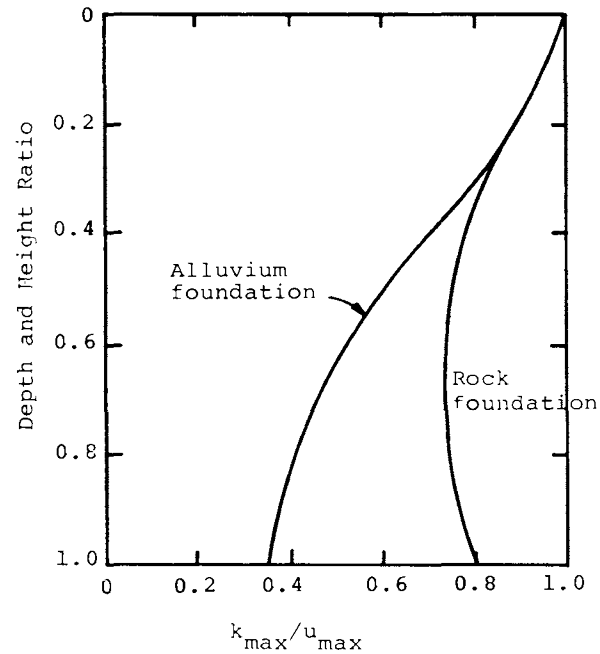

Figure 2. Variation of Average "Maximum Acceleration Ratio" With Depth of Sliding Mass.

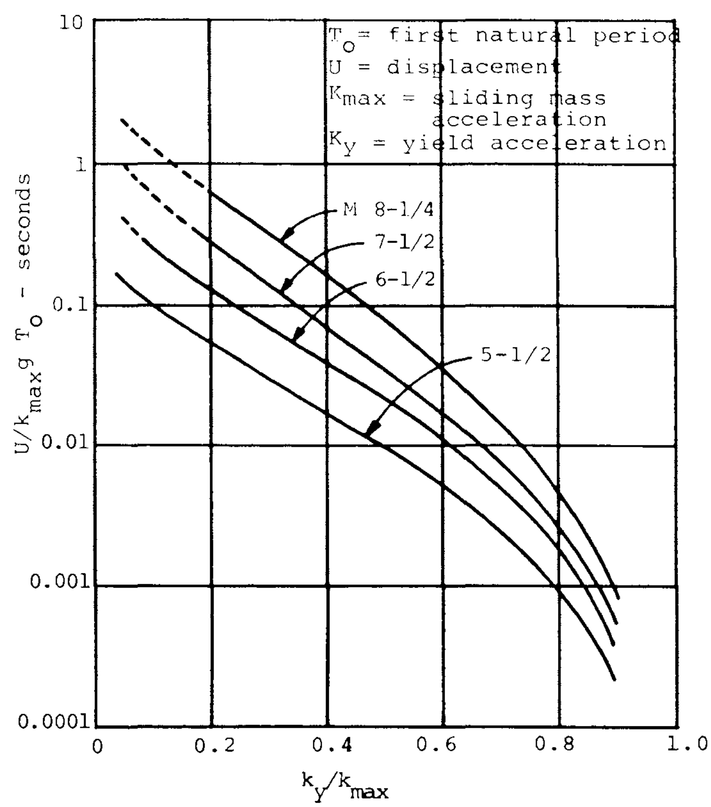

Figure 3. Variation of Average Normalized Displacement With Yield Acceleration.

$$\dot{X}_t = a + \frac{\Delta t}{2}\ddot{X}_t \qquad (8)$$

$$X_t = b + \frac{\Delta t^2}{6}\ddot{X}_t \qquad (9)$$

where

$$F = \left[1.0 + \frac{\Delta t}{2}(2\lambda\omega) + \frac{\Delta t^2}{6}\omega^2 \right]^{-1} \qquad (10)$$

$$a = \dot{X}_{t-\Delta t} + \frac{\Delta t}{2}\ddot{X}_{t-\Delta t} \qquad (11)$$

$$b = X_{t-\Delta t} + \Delta t\,\dot{X}_{t-\Delta t} + \frac{\Delta t^2}{3}\ddot{X}_{t-\Delta t} \qquad (12)$$

The solution of Equation 6 by step-by-step method proceeds as follows: The initial displacement X_o and the initial velocity, $\dot{X}_o$ are given as the initial conditions of the problem. The initial acceleration is obtained from Equation 6 as:

$$\ddot{X}_o = \ddot{U}_g(0) - 2\lambda\omega\,\dot{X}_o - \omega^2 X_o \qquad (13)$$

Then the step-by-step response of the system is obtained by repeated application of Equations 7, 8 and 9. During these application, the sliding mass acceleration $\ddot{X}_{tg}$ is checked with $\ddot{X}_{yn}$. If $\ddot{X}_{tg}$ is greater than or equal to the yield acceleration, $\ddot{X}_{yn}$, $\ddot{X}_{tg}$ is set to equal to $\ddot{X}_{yn}$. In order to simulate gradual decrease in shear strength of soil under dynamic loadings due to the pore pressure built-up when the acceleration $\ddot{X}_{tg}$ exceeds the yield acceleration, $\ddot{X}_{yn}$ is defined as:

$$\ddot{X}_{yn} = \delta n\,\ddot{X}_{y\,max} \qquad (14)$$

where, $\ddot{X}_{ymax}$ is the maximum yield acceleration obtained from the pseudo-static analysis of embankment and δn is the shear strength reduction factor for the n cycle.

The permanent displacement was taken to be the difference between displacement spectrum values of the non-linear and linear systems. The basic steps required in the computations are:

a) Determine the yield acceleration from the pseudo-static stability analysis as proposed by Makdisi and Seed (1978).

b) Determine the average maximum sliding mass acceleration K_{max} from Figure 2 as recommended by Makdisi and Seed (1978).

c) Develop the acceleration and displacement spectrum curves from the step-by-step integration as proposed in this paper for an earthquake record selected for the dam site. These curves for the given examples are shown in Figures 4, 5 and 6.

d) Draw a line horizontally of K_{max} until it intersect the acceleration spectrum curve and then draw a vertical line until it intersect the permanent displacement spectrum curve. The displacement of the intersection point is the computed permanent displacement for the sliding mass as shown in Figures 4, 5, and 6.

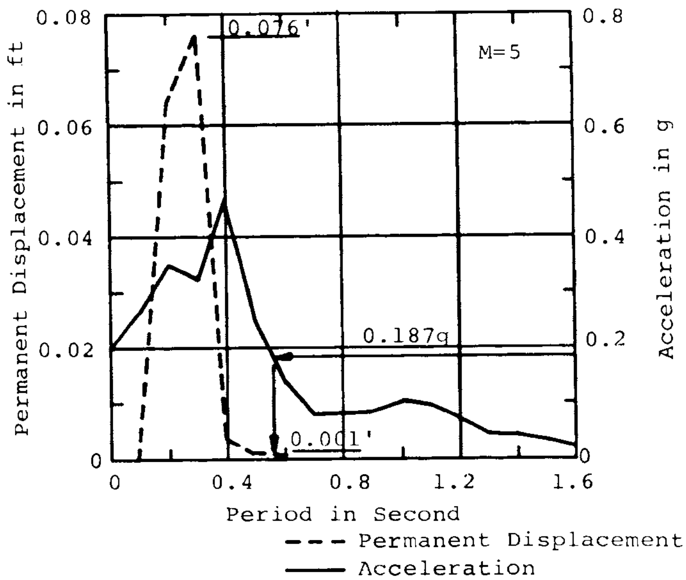

Figure 4. Guri Main Dam Permanent Displacement And Acceleration Response Spectra

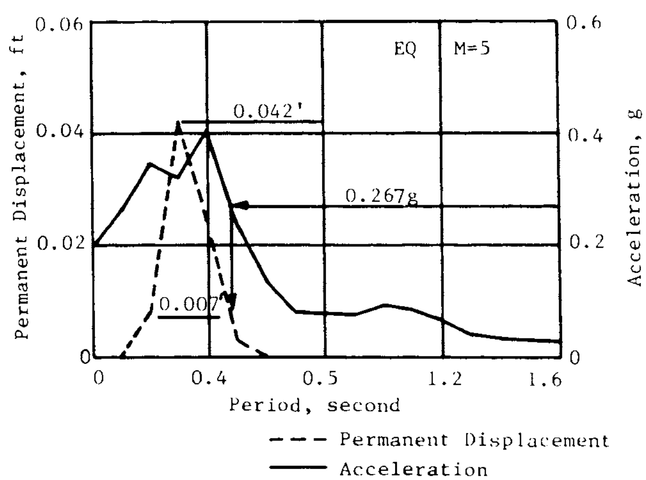

Figure 5. Guri Dike Permanent Displacement and Acceleration Response Spectrum

EXAMPLES

The permanent displacement of Guri Main Dam, dike and La Honda Dam in Venezuela produced by earthquakes were computed with the above described procedures. The pseudo-static slope stability analyses were performed with various seismic coefficients for each sliding mass to establish the yield acceleration for the sliding mass. The dynamic analyses of these dams were also made to determine the response acceleration at various points in dams under the selected base earthquake motion at sites. The pertinent data regarding these dams for the evaluation are summarized in Table 2. The summary of the results on permanent deformation obtained from various methods are tabulated on Table 3.

The Guri main dam and the Guri dike were analyzed by using Parkfield-Temblor No. 2 N65W Earthquake scaled to a peak acceleration of 0.2g. The La Honda dam was analyzed by using the Pocoima and Taft earthquake record scaled to a peak acceleration of 0.65g. The crest settlements of Guri main dam and dike computed by the methods used herein ranged from 0.03 cm to 6.1 cm for the critical deep surfaces. On other hand, for the critical shallow failure surface, the computed crest settlements of La Honda dam ranged between 0.72 m to 1.34 m. The results of permanent deformation analyses indicated that the values obtained from Newmark and Makdisi-Seed methods agree well with the maximum displacement obtained from the authors method. However, the author's values for the Kmax are considerably less than those obtained by the other methods. The difference of the results can be attributed to different earthquake records used in the analysis.

As shown in Figure 6, the maximum permanent displacement for La Honda dam was reduced from 1.05 m (3.45 ft) with the shear strength reduction to 0.735 m (2.41 ft) without the shear strength reduction. This indicates that the permanent displacement without the shear strength reduction is about two third of the placement with the shear strength reduction.

CONCLUSION

The results of the permanent displacement analysis showed that the permanent dam deformation could be predicted by various methods with the reasonable agreement on results. The maximum values of the displacement obtained from the permanent displacement response spectrum curve agree fairly well with those value obtained from Newmark and Makdisi-Seed methods. This means that for a given earthquake record with the yield acceleration of sliding mass, damping ratio and the fundamental period, it is possible to obtain the anticipated permanent displacement with the reasonable accuracy. The analytical results also showed that the shear strength reduction during the earthquake shaking would cause significant changes in permanent displacement as shown in Figure 6.

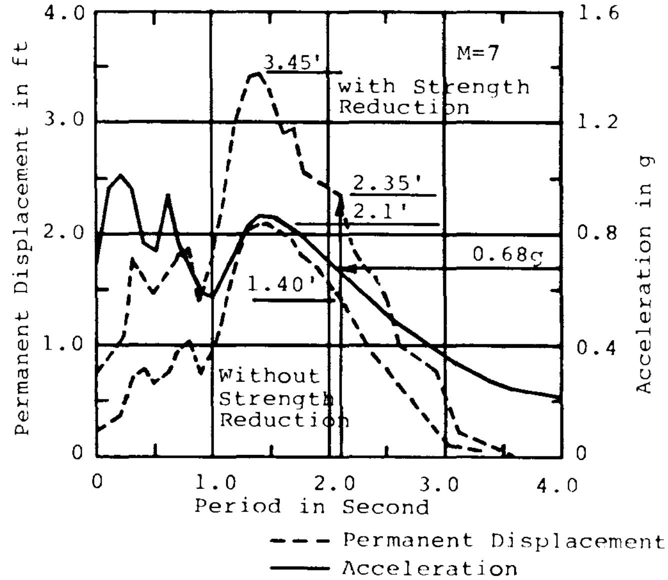

Figure 6. La Honda Dam Permanent Displacement And Acceleration Response Spectra

TABLE 2. Pertinent Data for Evaluation of Permanent Deformation

	Guri Earthfill Dam	Guri Dike	La Honda Dam
Dam Height, m	97	20	141
Slope (U/S)	3:1	3:1	3:1
TO_1, second	0.85	0.33	2.99
A_{max}, g	0.2	0.2	0.65
V_{max}, g	0.35	0.5	0.71
K_{max}, g	0.187	0.267	0.680
K_y, g	0.14	0.17	0.31
G_{ave}, ksf	3,847	2,210	820
V_s, fps	984	525	405
M	5.5	5.5	7.0
λ, %	10	13	18

TABLE 3. Summary of Predicted Permanent Displacement on Three Dams

	Newmarks	Ambrasey's Elastic plastic	Makdisi-Seed's Simplified	Author's Method For K_{max}	Author's Method Maximum
Guri Earthfill Dam	1.0 cm	0.20 cm	0.15 cm	.03 cm	2.2 cm
Guri Dike	6.0 cm	3.4 cm	1.2 cm	.2 cm	1.3 cm
La Honda Dam	1.03 m	N.A.	1.34 m	.72 m	1.05 m

Note: N.A. - not applicable

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