4th International Conference on
Earthquake Geotechnical Engineering
June 25-28, 2007
Paper No. ++++

A STUDY ON THE PSEUDOSTATIC ANALYSES OF THE UPPER SAN FERNANDO DAM USING FE SIMULATION AND OBSERVED DEFORMATIONS DURING THE 1971 EARTHQUAKE

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ABSTRACT

The pseudostatic approach is one of the available analytical techniques used for conducting the seismic slope stability analysis in the design and evaluation of earth structures. In this method, which is based on the limit equilibrium and stress–deformation analyses, the effects of an earthquake is represented by applying static horizontal and/or vertical accelerations to a potentially unstable mass of soil. Representation of the complex, transient and dynamic effects of earthquake shaking by a single constant acceleration is clearly crude and has significant shortcomings. The accuracy of the pseudostatic approach is governed by the accuracy with which the simple pseudostatic inertial forces represent the complex dynamic inertial forces that actually exist in an earthquake.

In this study, the behavior and performance of the San Fernando dam under the 1971 San Fernando earthquake has been studied and investigated. The finite element model of the dam was prepared based on the detailed available data and results of in-situ and laboratory material tests. Dynamic analyses were conducted to simulate the earthquake-induced deformations of the San Fernando dam using Plaxis software. Then the pseudostatic seismic coefficient used in the design and analyses of the dam was compared with the seismic coefficients obtained from dynamic analyses of simulated model and other proposed pseudostatic correlations. Based on the comparisons made, the accuracy and reliability of the pseudostatic analyses are evaluated and discussed.

Keywords: Pseudostatic Analyses, FE Simulation, San Fernando Dam

INTRODUCTION

The seismic stability of earth structures has been analyzed by pseudostatic procedures for many decades in which the effects of an earthquake are represented by constant horizontal and/or vertical accelerations. The first explicit application of the pseudostatic approach to the analysis of seismic slope stability has been attributed to Terzaghi (1950). In their most common form, pseudostatic analyses represent the effects of earthquake shaking by pseudostatic accelerations that produce inertial forces which act through the centroid of the failure mass. The magnitudes of the pseudostatic accelerations should be related to the severity of the anticipated ground motion. Selection of pseudostatic accelerations for design is not a simple matter and requires great care; values considerably smaller than the peak acceleration of the sliding mass are usually used.

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Stability is expressed in terms of a pseudostatic factor of safety calculated by limit equilibrium procedures. The pseudostatic approach can be used to evaluate pseudostatic factors of safety for planar, circular and noncircular failure surfaces. Many commercially available computer programs for limit equilibrium slope stability analysis have the option of performing pseudostatic analyses.

Limit equilibrium analyses consider force and/or moment equilibrium of a mass of soil above a potential failure surface. The soil above the potential failure surface is assumed to be rigid and the available shear strength is assumed to be mobilized at the same rate at all points on the potential failure surface. As a result, the factor of safety is constant over the entire failure surface. Because the soil on the potential failure surface is assumed to be rigid-perfectly plastic, limit equilibrium analyses provide no information on slope deformations.

The results of pseudostatic analyses are critically dependent on the value of the seismic coefficient which is defined as the ratio of the pseudostatic acceleration to the acceleration of gravity ($k_h = a_h / g$ and $k_v = a_v / g$). Selection of an appropriate pseudostatic coefficient (particularly k_h) is the most important, and the most difficult, aspect of a pseudostatic analysis. The seismic coefficient controls the pseudostatic force on the failure mass, so its value should be related to some measure of the amplitude of the inertial force induced in the potentially unstable material. If the slope material was rigid, the inertial force induced on a potential slide would be equal to the product of the actual horizontal acceleration and the mass of the unstable material. This inertial force would reach its maximum value when the horizontal acceleration reached its maximum value. In recognition of the fact that actual slopes are not rigid and that the peak acceleration exists for only a very short time, the pseudostatic coefficients used in practice generally correspond to acceleration values well below the maximum value. Terzaghi (1950) originally suggested the use of k_h=0.1 for sever earthquakes (Rossi-Forel IX), k_h=0.2 for violent and destructive earthquakes (Rossi-Forel X), and k_h=0.5for catastrophic earthquakes. Seed (1979) listed pseudostatic design criteria for 14 dams in 10 seismically active countries; 12 required minimum factors of safety of 1.0 to 1.5 with pseudostatic coefficients of 0.10 to 0.12. Marcuson (1981) suggested that appropriate pseudostatic coefficients for dams should correspond to one-third to one-half of the maximum acceleration, including amplification or deamplification effects, to which the dam us subjected. Using shear beams models, Seed and Martin (1966) and Dakoulas and Gazetas (1986) showed that the inertial force on a potentially unstable slope in an earth dam depends on the response of the dam and that the average seismic coefficient for a deep failure surface is substantially smaller than that of a failure surface that does not extend far below the crest. Seed (1979) also indicated that deformations of earth dams constructed of ductile soils with crest accelerations less than 0.75*g* would be acceptably small for pseudostatic factors of safety of at least 1.15 with k_h=0.10 (*M*=6.5) to k_h=0.15 (*M*=8.25). This criteria would allow the use of pseudostatic accelerations as small as 13 to 20 percent of the peak crest acceleration. Hynes-Griffin and Franklin (1984) applied the Newmark sliding block analysis to over 350 accelerograms and concluded that earth dams with pseudostatic factors of safety greater than 1.0 using $k_h = 0.5a_{\max} / g$ would not develop dangerously large deformations.

As can be seen from above discussions, there are no hard and fast rules for selection of a pseudostatic coefficient for design. However, it seems that the pseudostatic coefficient should be based on the actual anticipated level of acceleration in the failure mass and that it should correspond to some fraction of the anticipated peak acceleration. Although engineering judgment is required for all cases.

Representation of the complex, transient, dynamic effects of earthquake shaking by a single constant unidirectional pseudostatic acceleration is obviously quite crude. Even in its infancy, the limitations of the pseudostatic approach were clearly recognized. Terzaghi (1950), stated that “the concept it conveys of earthquake effects on slopes is very inaccurate, to say the least,” and that a slope could be unstable even if the computed pseudostatic factor of safety was greater than 1.0. Detailed analyses of historical and recent earthquake-induced landslides have illustrated significant shortcomings of the pseudostatic approach. Experience has clearly shown, for example, that pseudostatic analyses can be unreliable for soils that build up large pore pressures or show more than about 15% degradation of strength due to earthquake shaking. Results of pseudostatic analyses of some earth dams (e. g., Upper San Fernando dam, Lower San Fernando dam, Sheffield dam, and Tailing dam) show that pseudostatic analyses produced factor of safety well above 1.0 for a number of dams that later failed during earthquakes. These cases illustrate the inability of the pseudostatic method to reliably evaluate the stability of slopes susceptible to weakening instability. Nevertheless, the pseudostatic approach can provide at least a crude index of relative, if not absolute, stability.

Despite the above-mentioned limitations, the pseudostatic approach has a number of attractive features. The analysis is relatively simple and straightforward. Indeed, its similarity to the static limit equilibrium analyses routinely conducted by geotechnical engineers makes its computations easy to understand and perform. It produces a scalar index of stability (the factor of safety) that is analogous to that produced by static stability analyses. It must always be recognized, however, that the accuracy of the pseudostatic approach is governed by the accuracy with which the simple pseudostatic inertial forces represent the complex dynamic inertial forces that actually exist in an earthquake. Difficulty in the assignment of appropriate pseudostatic coefficients and in interpretation of pseudostatic factors of safety, coupled with the development of more realistic methods of analysis, have reduced the use of the pseudostatic approach for seismic slope stability analyses. Methods based on evaluation of permanent slope deformation are being used increasingly for seismic slope stability analysis.

UPPER SAN FERNANDO DAM SPECIFICATIONS

Upper San Fernando Dam Geometry and Performance

The Upper San Fernando dam located northwest of Los Angeles and north of the Lower San Fernando dam was built in 1922 using semi-hydraulic fill technique (Seed et al. 1973). The dam was about 24.4 meters high and was constructed upon 15.5 m of alluvial deposits overlying bedrock.

The 1971 San Fernando earthquake had a moment magnitude of 6.7 and an epicenter about 13 Km from the dam site. The peak horizontal acceleration at the dam site was estimated to be around 0.6g. Several longitudinal cracks were observed along almost the full length of the dam on the upstream slope slightly below the pre-earthquake reservoir level. The crest of the dam settled 0.76 m down and moved 1.5 m downstream. The maximum amount of horizontal displacements was about 2 m. Sand boils below the toe and increased water levels in three standpipe piezometers suggested that soil liquefaction had occurred. Water overflowed from two of piezometers. The reservoir level at the time of earthquake was at elevation 369.51 m, 1.83 m below the crest of the dam.

Properties of Upper San Fernando Dam

As shown in Fig. 1, the dam is divided into 9 different layers each representing different soil material zone. These soil units of dam section has been classified using equation (1) suggested by Sawada. The equation indicates the effect of height variation on the shear wave velocity and shear modulus of the layers materials located at different levels of the dam section.

$$V_S = 140 Z^{0.34} \tag{1}$$

where V_S is the shear wave velocity and Z is the dam height below the crest.

The relationship between shear wave velocity, shear and Young's moduli (G and E) are as follows:

$$G = r \times Vs^{2} \tag{2}$$

$$G = \frac{E}{2(1+n)}$$

(3)

where ν is the Poisson's ratio.

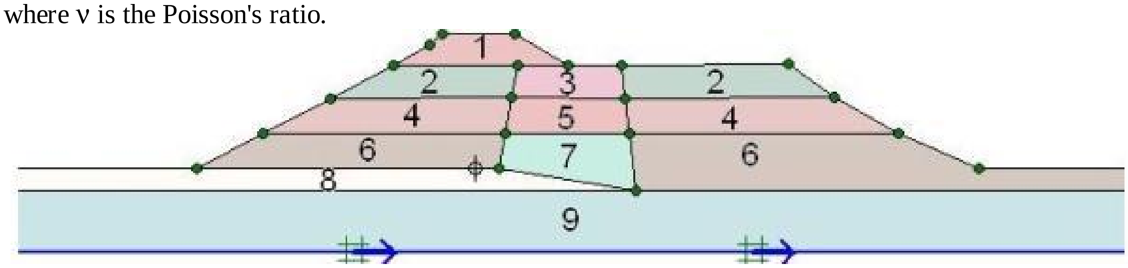


Figure 1. Cross section and layers of Upper San Fernando dam

The layers' soil parameters associated with the nine soil zones obtained using the above equations and detailed results given by Seed et al. (1973) are listed in Table 1.

Table 1. Soil properties of the Upper San Fernando dam

Layer No.	g_{dry} [kN/m^3]	g_{wet} [kN/m^3]	E [MN/m^2]	V_S [m/s]	c [kN/m^2]	j [$degree$]	n	K [m/day]
1	20.6	22	376.6	252	5.124	28	0.4	0.06
2	18	19.2	198	200	2	37	0.35	0.06
3	17.9	19.2	197	200	2	37	0.35	0.06
4	18	19.2	340	262	2	37	0.35	0.06
5	17.9	19.2	338	262	2	37	0.35	0.06
6	18	19.2	544	330	2	37	0.35	0.06
7	17.9	19.2	570	340	2	37	0.35	0.06
8	19	20.3	539	315	2	37	0.4	0.06
9	19	20.3	1640	550	2	37	0.4	0.04

STUDY METHODOLOGY

Most engineers consider the seismic coefficient as a means of designating the magnitude of a static force which is equivalent in effects (i.e. produces the same deformations of the earth dam) to the actual dynamic inertia forces induced by the earthquake. But how would the seismic coefficient denoting this equivalent static force be determined? It would seem that the determination of an appropriate value would necessarily involve two steps:

1) Determination and specification of deformations and degree of instability of dam induced by the earthquake, and

2) Evaluation of equivalent static force with the capability to make the same displacements or instabilities.

It would appear that any attempt to select a final value of such a seismic coefficient without going through step (1) and without a large back-log of experience to guide the selection could have little reliable basis.

In order to determine exact results for stage (1), it will be preferable to utilize dynamic analyses based on finite element method, and hence the Plaxis software seems to be an appropriate choice. High accuracy of dynamic analysis puts it at high point of view. The results obtained from two-dimensional dynamic analysis of Upper San Fernando dam under San Fernando 1971 earthquake, such as horizontal and vertical displacements, almost justify the observed displacements. Then an equivalent static force is determined for each layer and seismic coefficient is obtained for those layers. In order to reach this aim, the static forces were activated to each layer's gravity center (as shown in Fig. 2) and displacements and dam deformations were gained. Fig. 3, for instance, shows the horizontal displacements of dam using the pseudostatic analysis which is well justified by the results of dynamic analysis shown in Fig. 7. The importance of this study shines in evaluating varying seismic coefficient for San Fernando dam and that is relevant to differentiation of each layer's coefficient. Assuming a constant seismic coefficient would be applicable for rigid structures and using this current method for earth dams which have not rigid-body response is not rational. Destruction of Lower San Fernando dam and Oshima Tailing dam confirm the invalidity of pseudostatic analysis with constant coefficient, since both of them had been designed using pseudostatic analysis having seismic coefficient of 0.15 and 0.2 respectively. In this study, the equivalent seismic coefficients for different soil zones of Upper San Fernando dam have been determined using two-dimensional dynamic analyses and large back-log, and then the results have been compared with the design seismic coefficient (0.15) of the dam.

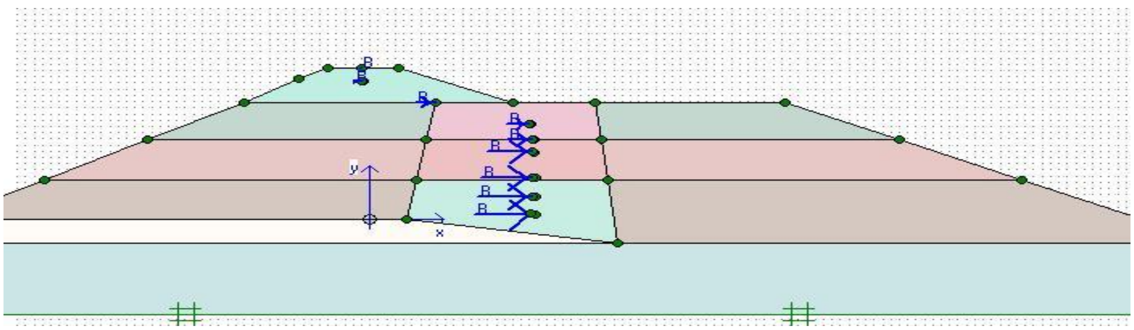


Figure 2. Equivalent static forces acting at layers gravity center

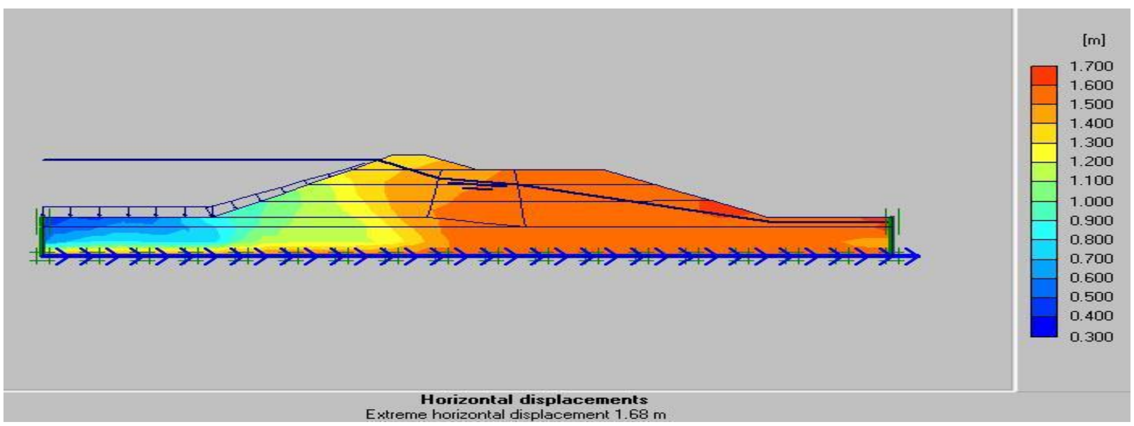


Figure 3. Pseudostatic horizontal displacements of dam

DYNAMIC ANALYSIS

Most of the problems encountered in the area of geotechnical engineering such as retaining walls, tunnels, earth dams and embankments are studied using two-dimensional dynamic analyses based on the finite element method (FEM) which is one of the available powerful numerical methods. The fundamental stages required to create a FE model include: selecting an appropriate element, dividing the model into elements and nods, extending equations of each element and determining element's stiffness matrix, combining element's matrix and creating a single matrix for model. Elements movement equation is given by:

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + [K]\{U\} = \{R(t)\} \quad (4)$$

in which [*M*] is the whole mass matrix, [*C*] is the whole damping matrix, [*U*] is the model nods axial movement, and {*R(t)*} is the axial force of model points.

One of the current methods used to solve the movement equation is Newmark step-by-step method. Newmark provided this method for dynamic analysis of earthquake loading. In this method displacement and velocity are determined using the following equations:

$$u_{t+\Delta t} = u_t + \dot{u}_t \Delta t + [(\frac{1}{2} - a)\ddot{u} + a\,\ddot{u}_{t+\Delta t}]\Delta t^2 \tag{5}$$

$$\dot{u}_{t+\Delta t} = \dot{u}_t + [(1-b)\ddot{u}_t + b\,\ddot{u}_{t+\Delta t}]\Delta t \tag{6}$$

where Δt is time pace and α and β are controlling parameters for numerical integration accuracy, according to the implicit Newmark scheme. In order to obtain a stable solution, these parameters have to satisfy the following condition:

$$\beta \geq 0.5 \qquad ; \qquad \alpha \geq 0.25(0.5 + \beta)^2 \tag{7}$$

Considering standard values (β=0.5) which is called average acceleration method, leads the calculations to rational results. Despite Newmark's damping method, taking advantage of β=0.6 and α=0.3025 values, in this study average acceleration method is being used to solve movement equations, as well as Newmark's method.

Special boundary conditions have to be defined in order to avoid the spurious reflections of the waves on the model boundaries. These boundaries are based on the Lysmer-Kohlmeyer model. According to this model, the normal and shear stress components absorbed by a damper are determined as follows:

$$s_n = -c_1\, r\, V_P\, \dot{u}_x \tag{8}$$

$$t = -c_2\, r\, V_S\, \dot{u}_y \tag{9}$$

where ρ = mass density, V_S = shear wave velocity, V_P = longitudinal wave velocity, $\dot{u}_x$ and $\dot{u}_y$ = velocity of particle motion in the direction of x and y, respectively. c_1 and c_2 are relaxation coefficients used to improve the wave absorption on the absorbent boundaries. c_1 corrects the dissipation in the direction normal to the boundary and c_2 in the tangential direction. The research and experience findings recommend to choose c_1=1 and c_2 = 0.25 for best results (Brinkgreve and Vermeer, 1998).

DAM SIMULATION

The process begins with specifying the clusters (9 clusters) and defining the properties relevant to each cluster. Fig. 4 shows the generated mesh section and also Fig. 5 exhibits upstream water level and phreatic line.

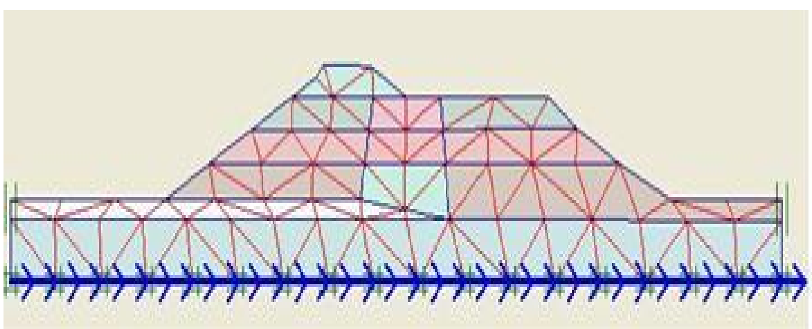


Figure4. Section of generated mesh of Upper San Fernando dam

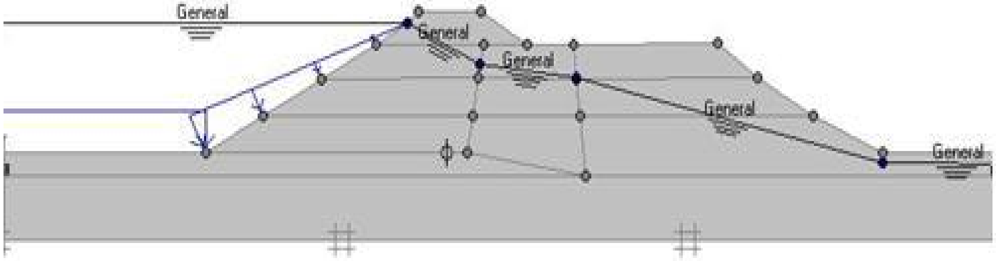


Figure 5. Upstream water level and phreatic line

The Numerical calculations using Plaxis software involve 3 Phases. First phase is dam plastic analysis conducted for the time when the construction is over. Second phase includes dam plastic analyses under own body load and finally the last step consists of dynamic analysis under San Fernando 1971 earthquake loading. The third phase loading is applied in the form of a file (accelerogram) input to the program. The whole deformations, horizontal and vertical displacements obtained in the output of the program are illustrated in Figs. 6 to 8.

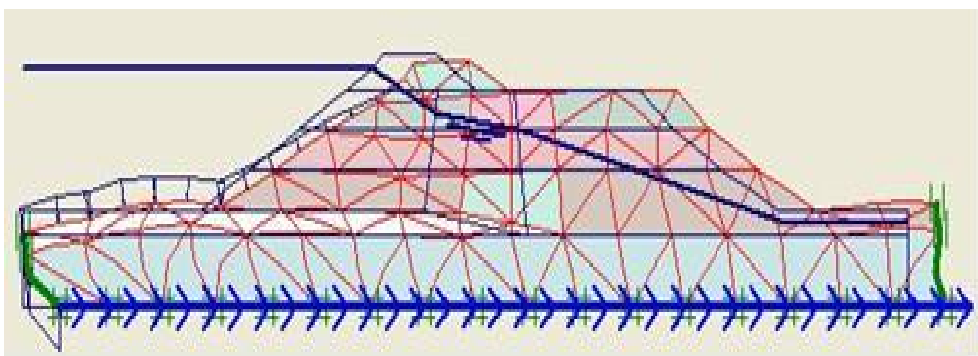


Figure 6. Whole deformations of dam

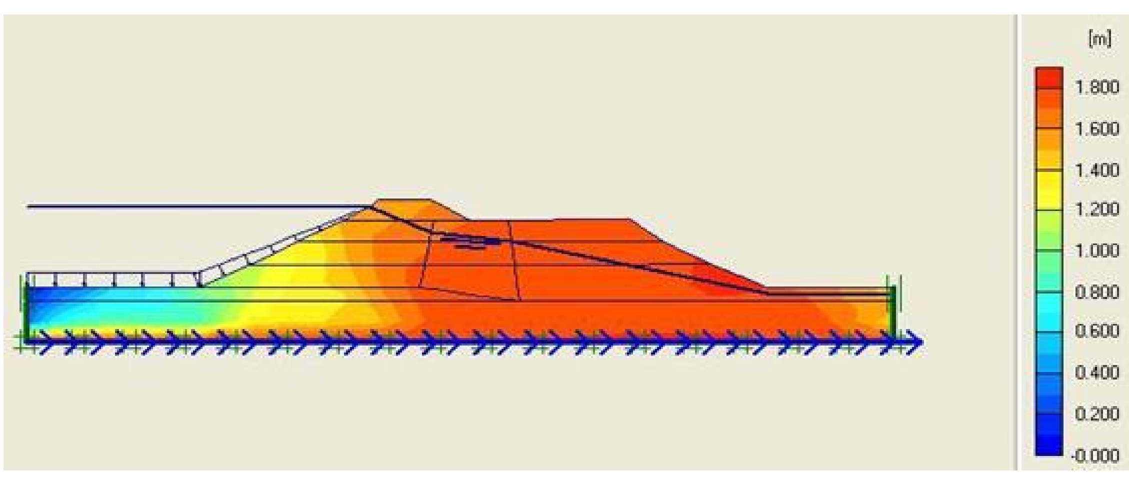


Figure 7. Horizontal displacements of dam

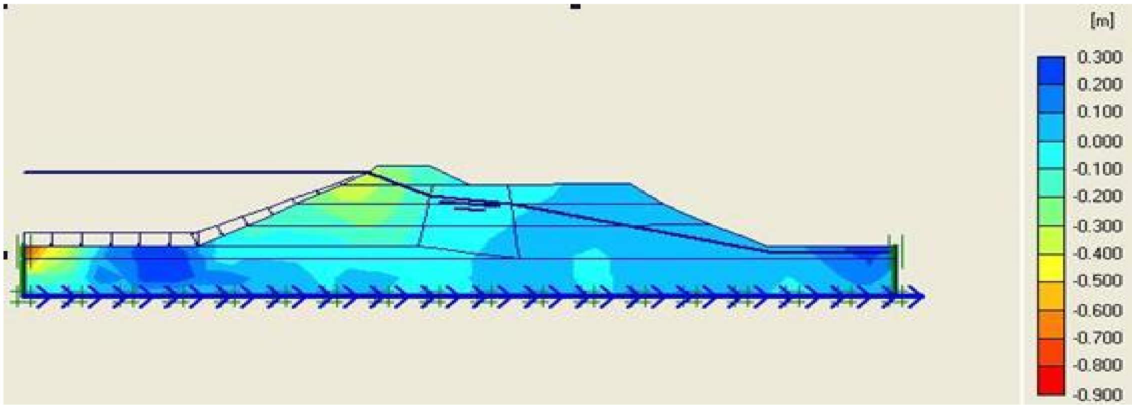


Figure 8. Vertical displacements of dam

STRESS-TIME ANALYSIS

Before starting calculation step, stress points are chosen on cross section of the dam. These points are located in the *x* direction with three points at each level as shown in Fig. 9 (one point upstream side, one point middle part, and one point downstream side). 27 points constituting 9 lines parallel with the *x* direction are specified. Stress (s_{xx})-time curves (27 curves) can be obtained from CURVE step of the program. Owing to the generation of numerous curves and for the sake of space saving, only some stress-time curves related to points at various levels of the sectio are provided. Figs. 10 to 21 show these envisaged curves.

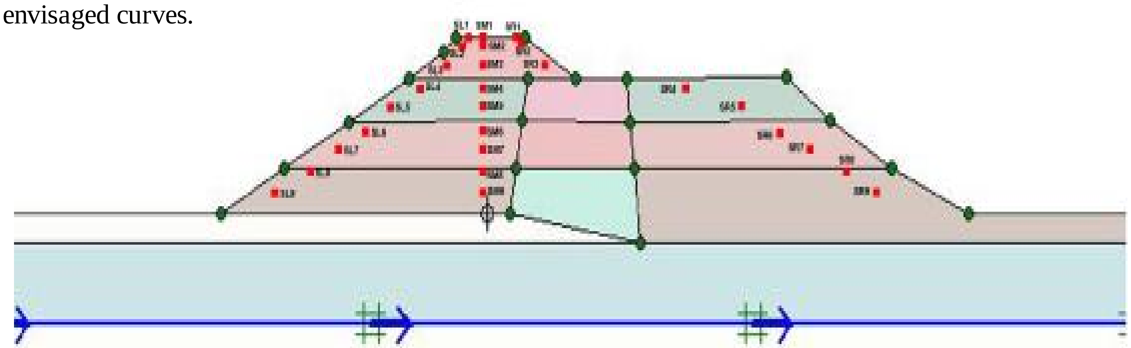
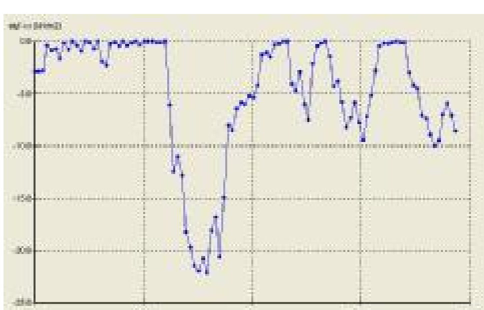


Figure 9. Location of stress points for curves



Time (s)

Figure 10. Stress-time curve for point SM1

Time (s)

Figure 11. Stress-time curve for point SM2

Time (s)

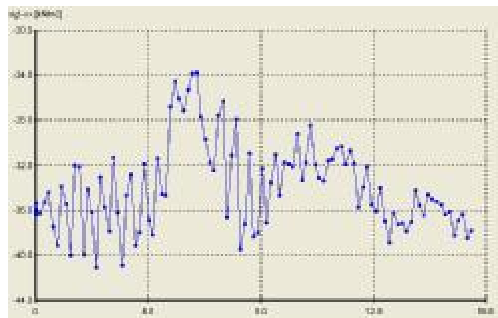
Figure 12. Stress-time curve for point SM4

Time (s)

Figure 13. Stress-time curve for point SM6

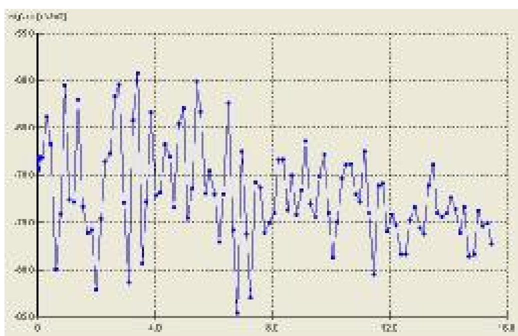
Time (s)

Figure 14. Stress-time curve for point SR2



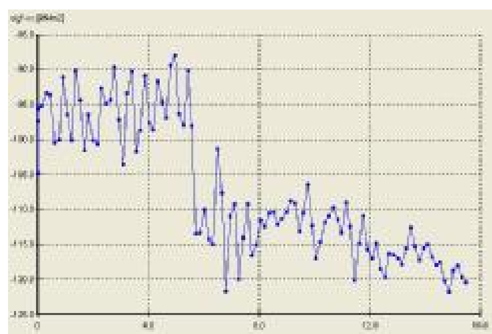
Time (s)

Figure 15. Stress-time curve for point SR4



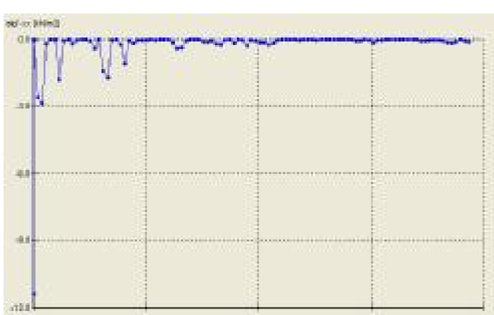
Time (s)

Figure 16. Stress-time curve for point SR7



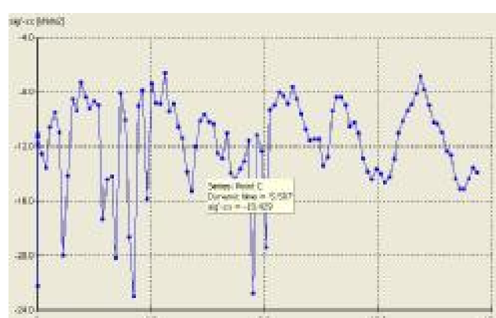
Time (s)

Figure 17. Stress-time curve for point SR9



Time (s)

Figure 18. Stress-time curve for point SL1

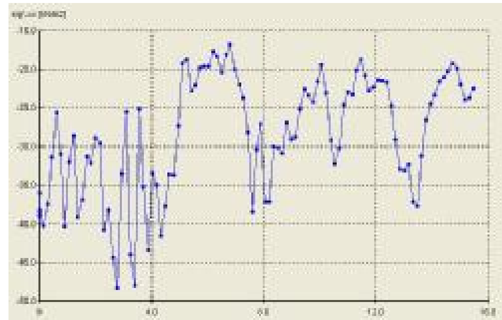


Time (s)

Figure19. Stress-time curve for point SL4

Time (s)

Figure 20. Stress-time curve for point SL7



Time (s)

Figure 21. Stress-time curve for point SL9

For each curve a maximum value is deliberated for a period and is considered due to its conservative value. Though the maximum value of each curve is multiplied by 0.7 then distribution of stress along the height of model is approximated to be linear for all points located in the upstream, downstream and middle parts. The results are shown in Figs. 22 to 24.

In addition, the equivalent force for each part of layer across stress point is determined using the product of height of layer to approximated stress value. The results are illustrated in Figs. 25 to 27.

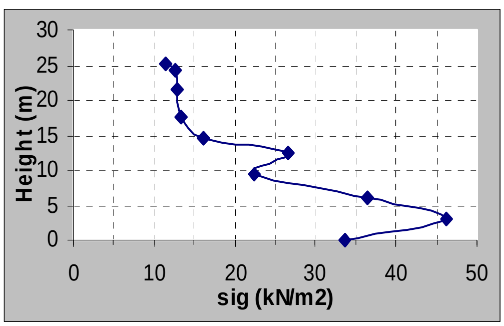


Figure 22. Approximated stress curve for upstream part of dam

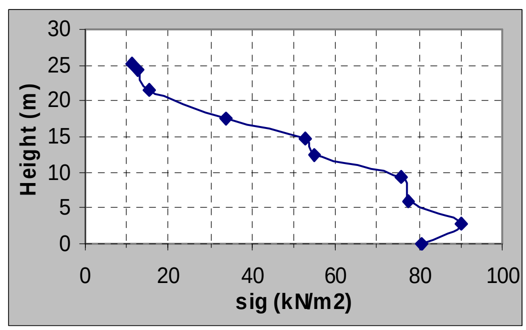


Fig.23. Approximated stress curve for middle part of dam

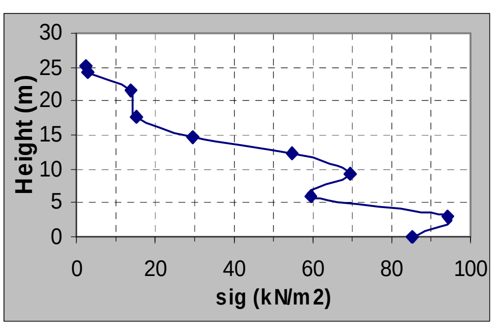


Figure 24.Approximated stress curve for downstream part of dam

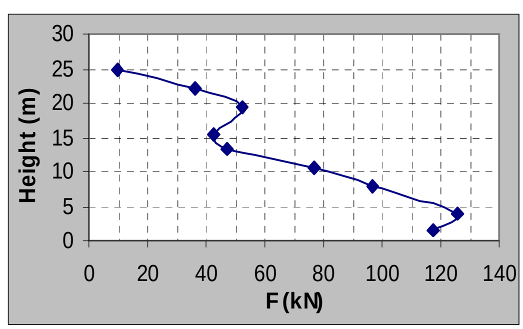
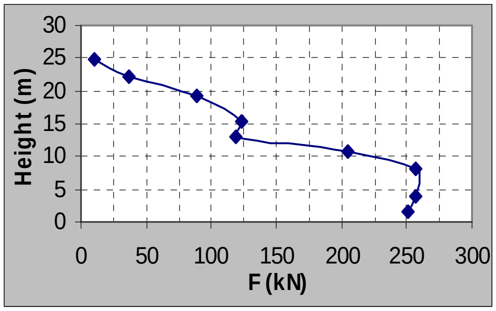


Figure 25. Equivalent force curve for upstream part of dam



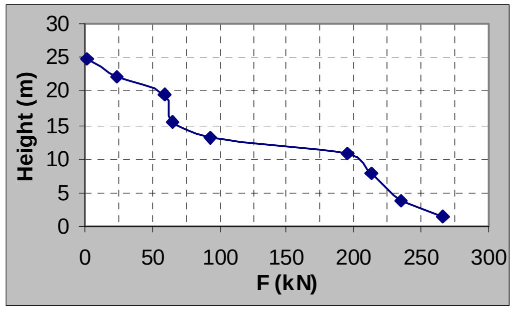


Figure 26. Equivalent force curve for middle part of dam

Figure 27. Equivalent force curve for downstream part of dam

Then each layer's equivalent force can be determined by employing the following equation:

$$F = \frac{\sum_{i=1}^{3} F_i L_i}{L} \qquad (10)$$

in which F_i is force of the part (upstream, middle, downstream), L_i is the effective length of the part, and L is the layer's length. Finally seismic coefficient is calculatedd by dividing layer force to its weight. The results are summarized and listed in Table 2.

Table 2. Seismic coefficients for Upper San Fernando dam under 1971 Earthquake loading

Layer No.	Force $\overline{F}\,(kN)$	Height of layer (m)	Weight W (kN)	Seismic coefficient $C = \overline{F}/W$
1	74	0.8	247	0.311
2	319.2	2.8	1205.4	0.272
3	675.4	4	3023.2	0.254
4	780.5	2.9	3252	0.241
5	870	2.2	3766	0.236
6	1530	3.14	6486	0.235
7	1712.3	3.3	7354.8	0.232
8	1811.1	3.06	7879.2	0.229
9	1895.3	2.95	8739.5	0.216

CONCLUSIONS

The results obtained from the analyses conducted for investigating the Upper San Fernando dam behavior under San Fernando 1971 earthquake loading indicate that the seismic coefficient increases with increasing of the height. The ratio of seismic coefficient at the crest over seismic coefficient at the base of the dam is about 1.44. The design seismic coefficient was 0.15 but the minimum calculated seismic coefficient for lower layer of this dam is 0.21. In this case the crest of the dam settled 0.76 m and moved 1.5 m downstream. The maximum amount of horizontal displacements was about 2 m. The results indicate that the constant seismic coefficient used in designing of Upper San Fernando dam was not applicable and in case of using constant seismic coefficient, it must be between the minimum value of 0.21 and the maximum value of 0.311. The following results are gained by comparing the design seismic coefficient to the calculated seismic coefficients:

- The maximum value of seismic coefficient offered by USBR is 0.2 while this coefficient is not applicable in the case of this dam.

- Although the minimum acceleration submitted by seismograph has a value about 0.34g but considering the rigid body response method (k=0.34) in this case which has a maximum seismic coefficient of 0.31 is extremely conservative.

- The maximum and minimum values determined using Ambraseys's method are 0.33 and .023 respectively. It seems that the rational value for seismic coefficient is existed between these evaluated values.

- This study shows that considering inconstant seismic coefficient in earth dam design is more realistic and rational than considering a constant seismic coefficient. Furthermore the procedure employed in this study can be utilized for evaluation of design seismic coefficient of constructed earth dams designed using pseudostatic analyses.

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