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SYNOPSIS: A model for the computation of seismically induced permanent displacements in earth structures is proposed. This model allows for relative sliding between different zones of the structure under investigation. Large displacements are allowed between these zones. Elasto-plastic constitutive soil behavior is accounted for. An Updated Lagrangian Jaumann Finite Element formulation is used. The proposed model is used to predict the observed behavior of La Villita dam during the November 15, 1975 earthquake.

INTRODUCTION

The main damage mechanisms in earthdams due to earthquakes are longitudinal and transversal cracking, shear by fault displacement, foundation failure and / or settlement, and sliding and shear distortion. A comprehensive study of more than 105 earthdams which were damaged due to earthquakes was performed [Seed et al, 1978; Woodward-Clyde Consultants, 1987] to investigate the relative importance of each of these damage modes. It was found that more than 70 % of these dams were damaged as a result of cracking, sliding and shear distortion. There are two types of failure mechanisms associated with shear distortion and sliding. The occurrence of either of these mechanisms depends on the extent to which pore water pressure builds up in the embankment during earthquake loading. These mechanisms are [NRC, 1985]: 1) parts of the dam or foundation may liquefy resulting in failure of the liquefied zone. Loose to medium dense saturated sandy embankments where rapid pore pressure build-up is possible are susceptible to failure in this mode; 2) horizontal and vertical displacements of parts of the dam may develop and cracks may occur due to sliding of shallow or deep slices of the dam along failure surfaces. Generalized deformation and settlement may also occur without the formation of a specific failure surface. Embankments built of compacted cohesive clays, dry sands or saturated dense sands are susceptible to damage in this mode.

The more severe damage modes such as liquefaction leading to catastrophic failure are not likely to occur in recently built dams. However, even well built dams on strong foundations may undergo permanent deformation and hence loss of free board as a result of earthquake loading. The state of the art is thought to be deficient in predicting permanent deformation magnitudes in dams where total catastrophic flow failure is not anticipated [NRC, 1985]. The available approaches for the estimation of seismically induced permanent deformations in these situations are based on the following:

1) The seismically induced permanent strains within each element of soil constituting the dam are integrated and an estimate of the resulting permanent displacements is obtained. Permanent strains are estimated either by: a) utilizing a combination of linear -or equivalent linear- dynamic analysis and laboratory tests on representative soil elements (stresses from the dynamic analysis are applied on representative soil elements in the laboratory and the resulting strains are used to obtain the deformed shape of the structure) [Seed et al, 1973; Lee, 1974; Serff et al, 1976; Marr et al, 1981], or, b) by performing a nonlinear dynamic response analysis of the structure under investigation [Elgamal et al, 1984; Prevost et al, 1985; Stara-Gazetas, 1986; Lacy, 1986; see also Gazetas, 1987 for more details].
2) Permanent displacements are assumed to occur along well defined failure surfaces whenever the shear strength along these surfaces is exceeded and during the period over which it is exceeded (sliding block approach [Newmark, 1965]). The original Newmark sliding block model assumes a rigid sliding zone. Modifications to this model have since been proposed to account for effect of the dynamic response of the dam [Makdisi et al, 1978] (acceleration time histories from a dynamic analysis of the intact structure being analyzed are used as input to the sliding block analysis), and to account for pore pressure buildup [Sarma, 1975].

The previously described methods either assume that permanent displacements result from accumulation of permanent strains within individual soil elements without the occurrence of relative sliding within the structure being analyzed or assume that permanent displacements are the result of relative sliding (sliding zone is modeled as a rigid block) only. However, permanent strains within individual soil elements as well as relative sliding between parts of a structure separated by zones of intense straining could be taking place simultaneously during seismic loading. This is accounted for in finite element formulations based on the concept of a joint element [Goodman et al, 1968; Goodman, 1976; Toki et al., 1981, 1983] in which elements of zero thickness (double nodes) are distributed at prescribed sliding interfaces. The main problem with the joint element approach is that interaction between "double node" pairs continues even after the nodes are at a large distance from each other. Models based on the concept of joint elements may therefore deteriorate in accuracy as the relative interface displacements increase. The model proposed in this study allows for large relative displacements to occur along pre-defined interaction surfaces and accounts for the elasto-plastic constitutive behavior of soil. The applicability of this model is demonstrated by calculating the dynamic response of a rigid yielding sliding block and that of a slope. The model is also used to estimate the observed permanent displacements in La Villita dam due to the November 15, 1975 earthquake.

MODEL FORMULATION

The structure under investigation may be divided into several interacting zones. The material inside each of these zones is modeled using finite elements. Interaction between the different zones is accounted for by using an interaction algorithm which enforces geometric compatibility at the interaction interfaces. A 2-Dimensional Updated Lagrangian Jaumann finite element formulation is used. This formulation allows for both geometric (large displacements) as well as material nonlinearities to develop. A brief discussion of the employed finite element formulation is described first followed by a description of the interaction algorithm.

The boundary value problem governing the dynamic response of a wide class of problems in soil dynamics is given by:

$$T_{ij,j} + b_i = \rho\ddot{u}_i \qquad \text{in } \Omega \tag{1}$$

subject to:

$$u = \bar{u}(x,t) \qquad \text{on } \Gamma_u$$

$$n.\tau = \tau^* \qquad \text{on } \Gamma_\tau$$

where:

τ_{ij} = generalized Cauchy stress tensor; b_i = body force, u_i = displacement, ρ = density, n = outward unit normal to Γ_τ, Γ_τ = part of domain where tractions are prescribed, Γ_u = part of domain where displacements are prescribed, $\Gamma_u \cap \Gamma_\tau = \phi$ and $\Gamma_u \cup \Gamma_\tau = \Gamma$, Ω = spatial domain occupied by the soil system, ϕ = Empty set, and a superimposed $(\dot{})$ denotes time differentiation.

The weak form of the equation of motion given by Eq. 1 is as follows [Belytschko and Hughes, 1983]:

$$\int_\Omega v_{i,j}\tau_{ij}d\Omega+\int_\Omega \rho v_i\ddot{u}_i d\Omega=\int_\Omega v_i b_i d\Omega+\int_{\Gamma_\tau} v_i\tau_i^* \, d\Gamma \tag{2}$$

in which: v_i = component of velocity vector ar it is assumed that v = 0 on Γ_u , $v_{i,j}$ velocity gradient.

The employed constitutive law relates the rat of deformation tensor to the Jaumann stress rat tensor as follows:

$$\overset{\circ}{\tau} = CD \tag{3}$$

where

$\overset{\circ}{\tau}$ = Jaumann stress rate tensor = $\dot{\tau}-\omega\tau+\tau\omega$ (4)

D_{ij} = Rate of deformation tensor = $(v_{i,j} - v_{j,i})/2$ (5)

ω_{ij} = spin tensor = $(v_{i,j}-v_{j,i})/2$ (6)

$\dot{\tau}$ = Cauchy stress rate tensor.

Finite element discretization is applied to the weak form of the equation of motion (Equation 2) and the resulting matrix equation is of the form:

$$M\ddot{U} = f^{\,ext} - f^{\,int} \tag{7}$$

where

M = Lumped mass matrix.

$\ddot{U}$ = vector of nodal accelerations.

$f^{\,int}$ = Vector of nodal internal forces obtained by assembling element internal force contributions.

$f^{\,ext}$ = Vector of nodal external forces obtained by assembling element external force contribution.

As indicated earlier, interaction between different zones constituting the structure being analyzed is modeled using an interaction algorithm. This interaction algorithm enforces geometric compatibility between interacting zones by prohibiting the penetration of nodes of one zone into elements of the other. This is accomplished by:

1- The location of all nodes constituting the interacting zones is determined assuming that there is no interaction between neighboring zones.

2- Penetration of nodes of each zone (slave nodes) into elements of the neighboring one (master elements) is checked. If any of the slave nodes penetrates into any master element, it is returned to the surface of this element. The resulting loss of momentum is transferred to the nodes of the master element into which the slave node has penetrated. This is accomplished according to the following procedure (Figure 1) [Lin, 1985]:

$$\bar{x}_f = a\bar{s} + \bar{x}_n \tag{8}$$

where

$$\bar{s} = (\bar{x}_p-\bar{x}_m)/|\bar{x}_p-\bar{x}_m|$$

$$a = \beta|\bar{x}_m-\bar{x}_n| \text{ if } \beta|\bar{x}_m-\bar{x}_n|<|\bar{x}_p-\bar{x}_m|$$

$$= |\bar{x}_p-\bar{x}_m| \quad \text{otherwise}$$

β = Coefficient of friction.

$\bar{x}_0$ = Position of slave node at time t − Δt.

$\bar{x}_n$ = Position of slave node at time t as calculated from finite element analysis before correction due to penetration.

$\bar{x}_m$ = Position of the point of intersection of normal to sliding surface drawn through $\bar{x}_n$.

$\bar{x}_f$ = Corrected position of slave node.

$\bar{x}_p$ = Position of point of intersection of line connecting $\bar{x}_0$ and $\bar{x}_n$ with sliding interface.

$\Delta\bar{r} = \bar{x}_f - \bar{x}_n$

and the change in velocity of the slave node is given by:

$$\Delta\bar{v}_s = \Delta\bar{r}/\Delta t$$

The updated velocity of the slave node is given by

$$\bar{v}_s = \bar{v}_s + \Delta\bar{v}_s$$

The loss of momentum associated with this adjustment is transferred to the nodes of the penetrated master element according to how far the particular node is from the final position of the slave node and such that the total linear momentum is conserved.

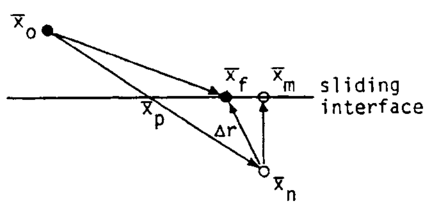


Figure 1. Procedure for repositioning a slave node [lin, 1985].

NUMERICAL EXAMPLES

1– Block On An Inclined Plane

The sliding block approach was proposed by Newmark [Newmark, 1965] for estimating seismically induced permanent displacements in earth structures. It is used to represent embankments or slopes subjected to earthquake loading where the applied cyclic stress plus the existing static stress exceed the shearing resistance of the soil along some postulated failure surface. A sample response obtained by such an approach is shown in in Figure 2 [Succarieh, 1990]. In this section, for illustration purposes, the sliding block response is calculated using the proposed model. The analysis is performed in two stages: gravity is applied pseudo-statically and is then followed by computation of the dynamic response. One element represents the sliding block while the slope is modeled using three elements (Figure 3). The following normalized material properties are used to model both the block as well as the underlying slope: interface coefficient of friction = 0.42; Poisson's ratio = 0.1; viscous damping ratio = 15 %; Young's Modulus = 1000000.0; and Δt = 0.005. The block–plane system was excited with a harmonic input of amplitude equal to 1.58 applied in the slope direction. The resulting response at node A of Figure 3 is shown in Figures 4 through 6. The deformed shape of the finite element mesh is shown in Figure 7. It can be seen from Figures 4 through 6 that the response is similar in character to that of the Newmark rigid block on an inclined plane (Figure 2): permanent displacements accumulate during the period over which the yield acceleration of the block is exceeded and the block acceleration response is biased with a transient (spike) occurring at the end of each sliding phase. Note that the value of yield acceleration (Fig. 6) and accumulated block displacements (Figure 4) are almost identical to the corresponding analytically derived value.

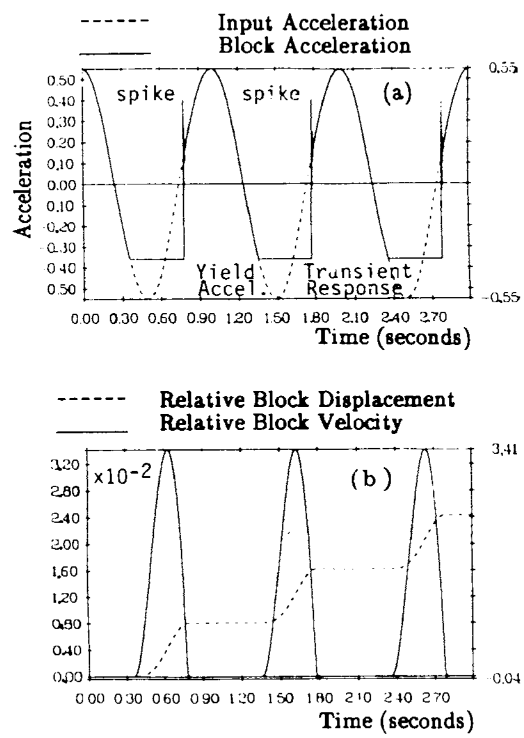


Figure 2. Block response: a) absolute acceleration; b) relative velocity and displacement.

2– Dynamic Response Of A Slope With One Weak Interface

The slope geometry is shown in Figure 8. The

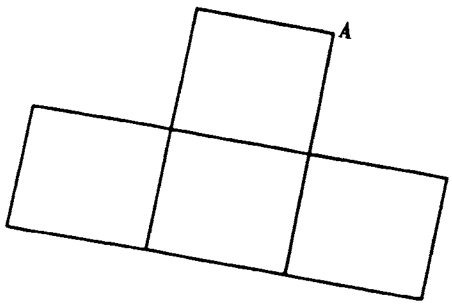


Figure 3. Block–slope finite element Discretization.

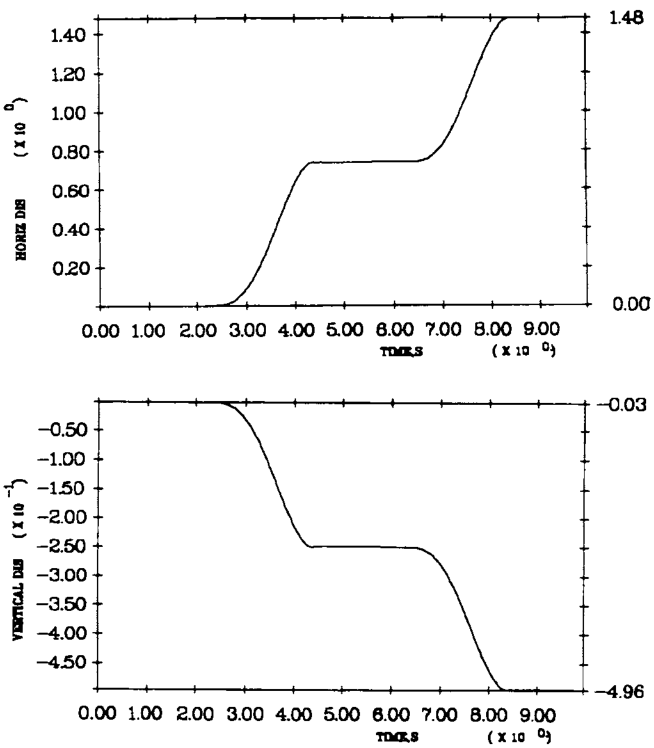


Figure 4. Block displacement response to a harmonic input motion (node A of Figure 3).

sliding zone of the slope is modeled using four elements separated by an interaction interface from the remaining soil. Angle of inclination of the slope is 26 degrees. Sand properties were used to represent the slope material: angle of internal friction = 36° ; density = 2.08 t/m^3; Young's modulus = 62000 t/m^2; and Poisson's ratio = 0.12. Gravity is applied pseudo–statically and Figure 8 depicts the resulting deformed shape. Input horizontal base excitation is applied in the horizontal direction with an amplitude of 10 m/s^{2}and the slope response is shown in Figures 9–11. The deformed mesh is shown in Figure 12. The resulting sliding zone response is very similar

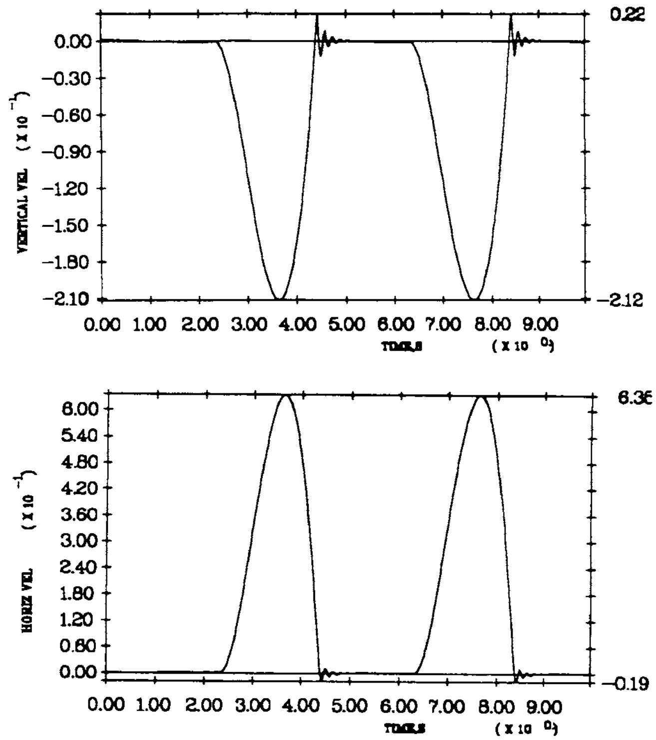


Figure 5. Block velocity response to a harmonic input motion (node A of Figure 3).

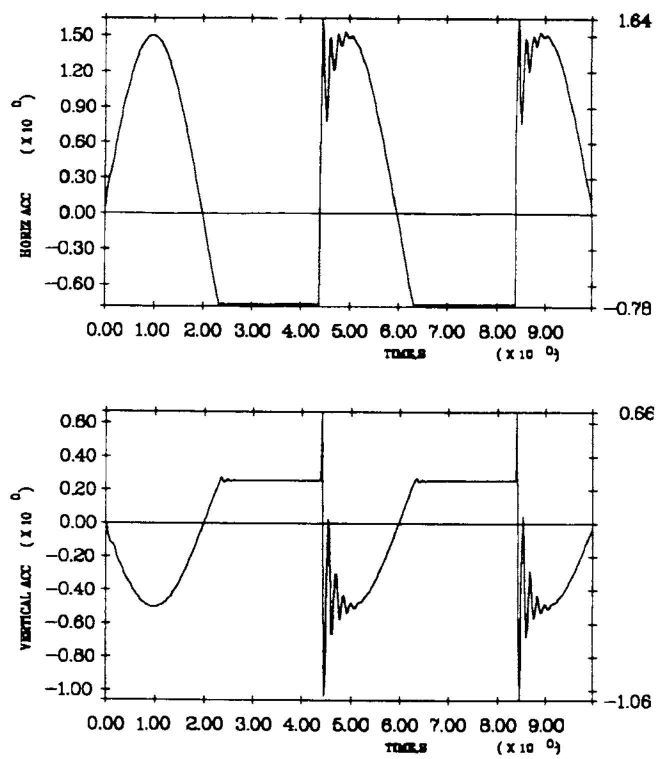


Figure 6. Block acceleration response to a harmonic input motion (node A of Figure 3).

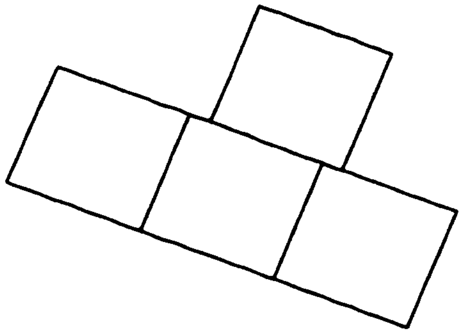


Figure 7. Deformed shape of finite element mesh.

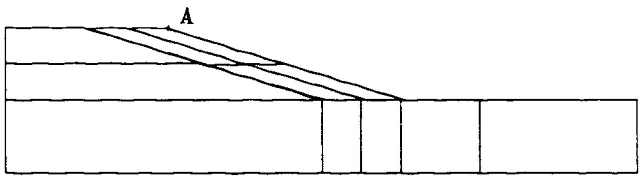


Figure 8. Finite element discretization of a slope.

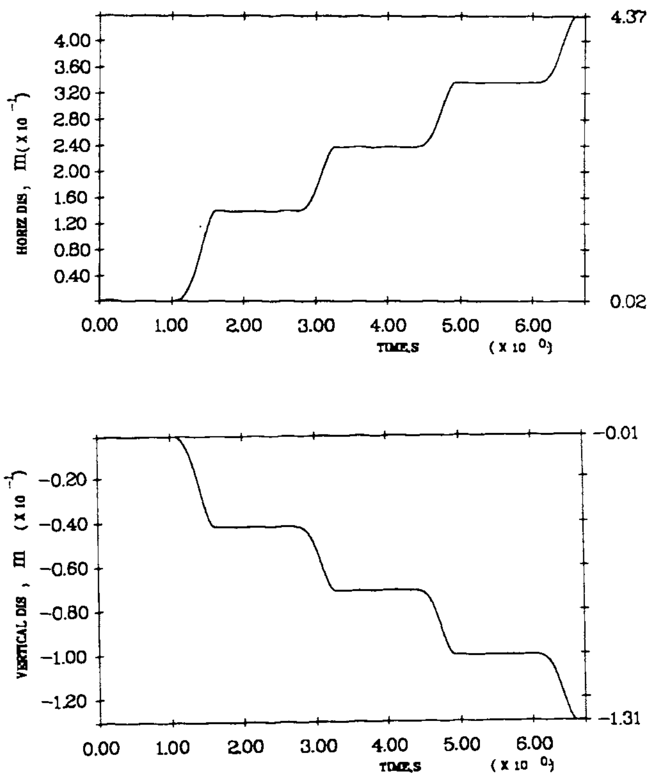


Figure 9. Sliding zone displacement response (node A of Figure 8).

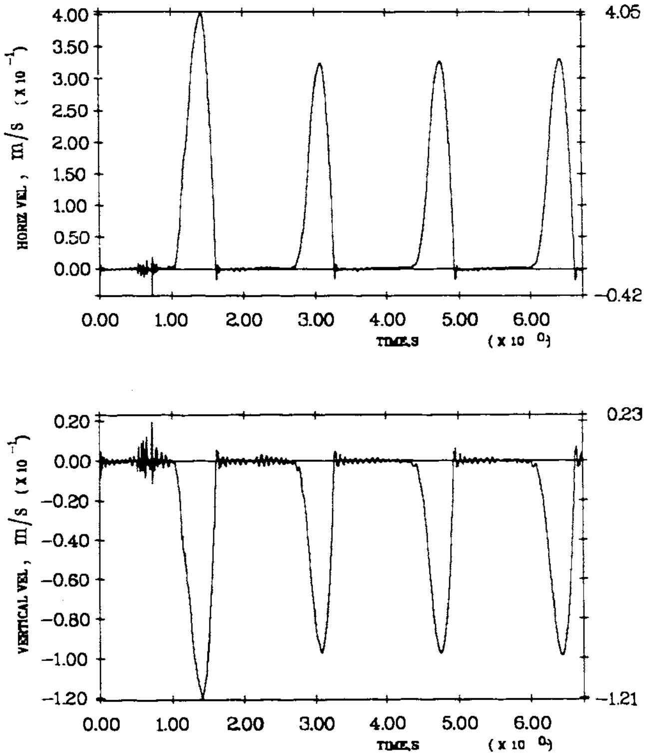


Figure 10. Sliding zone velocity response (node A of Figure 8).

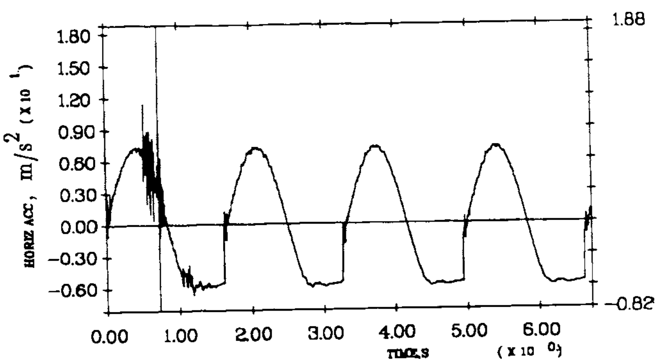


Figure 11. Sliding zone acceleration response.

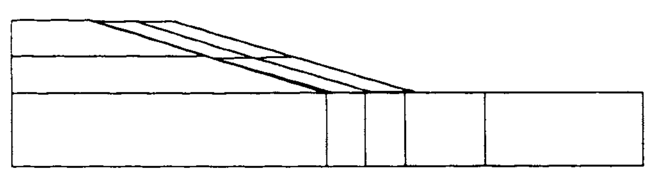


Figure 12. Deformed shape of slope mesh.

to that of the sliding block described in Example 1. Note that, for the slope being analyzed, a large amplitude horizontal acceleration (> 0.7g) is required to induce significant permanent displacements.

3- Response Of La Villita Dam

The proposed model is used in this section [Succarieh, 1990] to predict the observed behavior of La Villita dam during the November 15, 1975 earthquake (Figure 13). La Villita dam is an earth and rockfill dam located in Mexico 350 km southwest of Mexico city, Mexico. Since its construction the dam was subjected to five earthquakes which resulted in significant horizontal and vertical displacements. The acceleration responses at the dam crest and at nearby bedrock are available for the November 15, 1975 earthquake. During this earthquake the dam suffered a crest horizontal displacement of about 2 centimeters [Elgamal et al, 1990; Comision Federal De Electricidad; 1979]. The response of the dam to this earthquake is calculated herein. The finite element mesh used to discretize the dam and the corresponding material properties are shown in Figures 14 and 15 . Material properties were chosen so as to match the first natural frequency of the dam (1.26 Hz). One sliding interface (for illustration purposes) is used to separate the dam into a sliding and an intact zone. Material behavior inside both the sliding as well as the intact parts is modeled using an elastic-perfectly plastic conical failure surface. The displacement response at nodes 84 (intact structure) and 69 (sliding zone) is shown in Figures 16 and 17. The acceleration response at node 69 is shown in Figure 18 . The accumulated permanent displacements of (of the

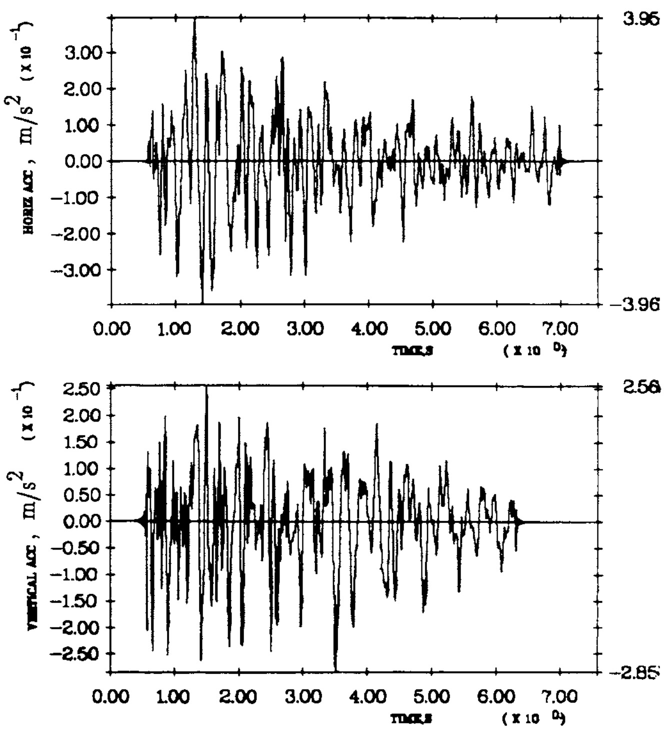


Figure 13. November 15, 1975 bedrock acceleration at La Villita.

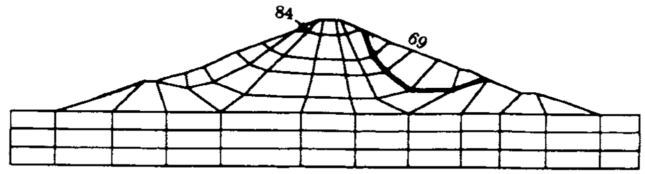
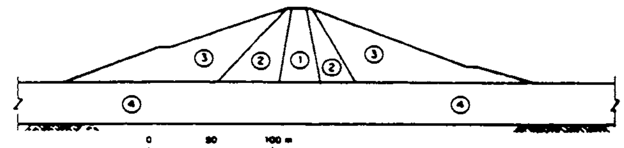


Figure 14. Finite element discretization of La Villita dam with one sliding interface.



Marerial	E, T/m^2	ν	γ, T/m^3
1	6.2X10^4	0.48	2.04
2	6.4X10^4	0.15	2.22
3	6.2X10^4	0.15	2.08
4	6.4X10^4	0.15	2.08

Figure 15. Material properties of different dam zones.

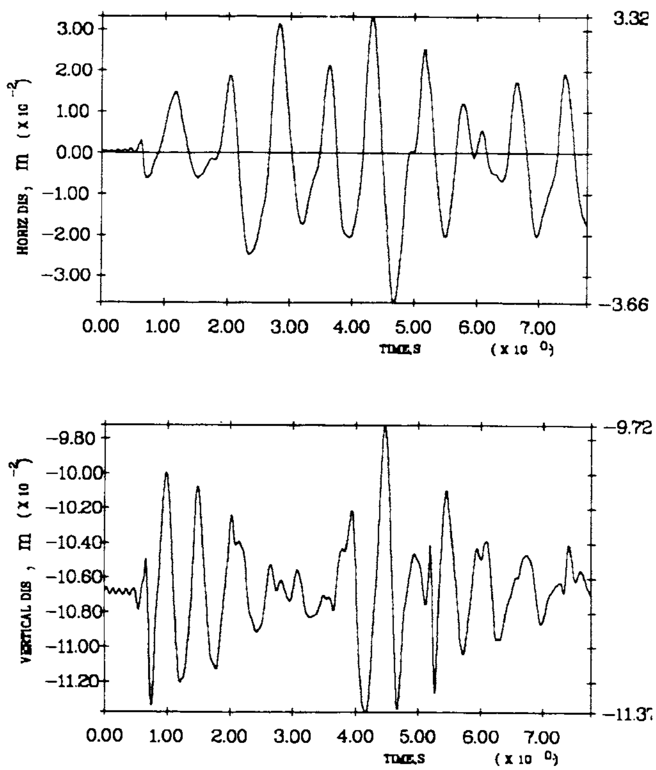


Figure 16. Displacement response at intact zone (node 84 of Figure 14) due to November 15, 1975 earthquake.

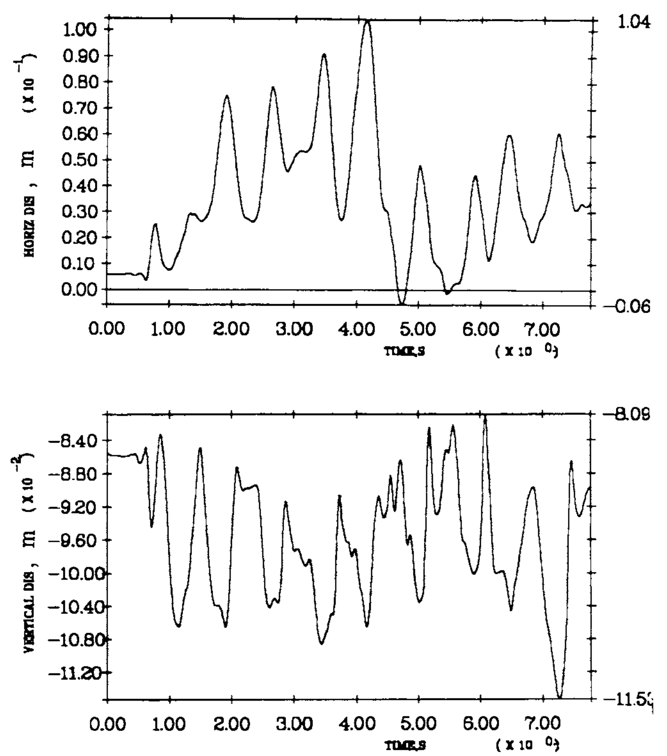


Figure 17. Displacement response at top of sliding zone (node 69 of Figure 14) due to November 15, 1975 earthquake.

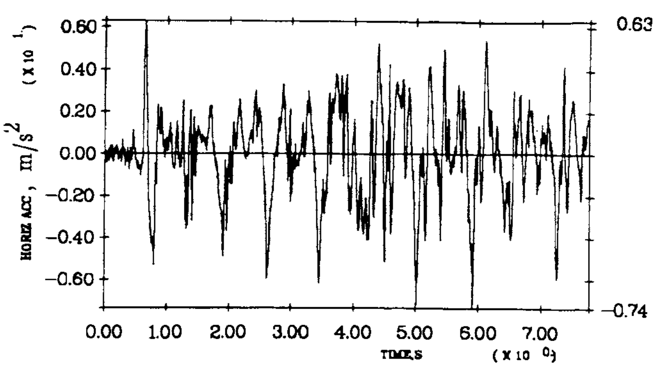


Figure 18. Acceleration response at top of sliding zone (node 69 of Figure 14) due to November 15, 1975 earthquake.

order of 1 cm) compare well to those actually observed.

SUMMARY AND CONCLUSION

A model is proposed for estimating seismically induced permanent displacements in earth structures. The proposed model allows for large relative sliding to take place between different parts of the analyzed structure during the calculation of dynamic response. Elasto-plastic soil behavior is also included. An Updated Lagrangian Jaumann finite element formulation is used. The ability of the model to predict permanent displacements is demonstrated by performing dynamic simulations of a rigid sliding block and a slope. For illustration purposes, this model is also used to calculate the observed behavior of La Villita dam during the November 15, 1975 earthquake. Calculated and observed displacements are found to be in reasonable agreement.

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