SEISMIC ANALYSIS OF EARTHDAMS USING A LAGRANGIAN PARTICLE METHOD.

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ABSTRACT :

Seismic safety of earth or rockfill dams and embankments is strongly conditioned by permanent displacements caused by earthquakes. For a severe earthquake, the permanent displacement pattern results from the combination of displacements generated by volumetric and shear plastic strains distributed within the structure, and those caused by sliding of a soil mass along a failure surface. Numerical procedures commonly used in practice do not consider the strain localization phenomena at failure surfaces and the associated mesh dependence of the solution. Typically, nonlinear finite element or finite difference codes yield an estimate of distributed deformations and dynamic response, without accounting for the plastic strain localization problem. In addition, some of the numerical approaches used in practice do not consider the configuration change caused by large displacements.
The material point method or MPM is a lagrangian "particle-mesh" numerical method. It has been previously used in modeling dynamic problems with large displacements and strain localization. With MPM, a body is discretized into a collection of lagrangian particles, which carry all the data needed to define the body's state. Interaction between particles takes place in a background fixed mesh, similar to those used in the finite element method.
The MPM is applied in this paper to model the dynamic response of a concrete faced gravel dam with increasing base movements. A comparison of the MPM displacement results with a simplified Newmark sliding block analysis is presented. The examples show that the MPM is a useful tool to assess the seismic safety of earth and rockfill structures.

KEYWORDS: MPM, plastic strain localization, earth dams

1. INTRODUCTION

The seismic security of earth and rockfill dams is strongly conditioned by the magnitude of the final displacements of the dam's body and its foundation after a destructive earthquake. For a very intense earthquake these permanent displacements are caused by the combination of displacements generated by volumetric and shear plastic deformation distributed within the structure, and sliding of a soil mass along one or several failure surfaces. The numerical procedures used in current practice usually do not consider the strain localization in these failure surfaces nor the dependence of the solution upon the size of the finite element or finite difference mesh. Most of the nonlinear dynamic codes currently used, yield only an estimate of the strains distribution and dynamic response, without adequately considering the localization of plastic deformations. In addition, some of the numerical procedures used in practice do not consider the changes in the configuration of the dam caused by large displacements.

A computer model designed to account for plastic strain localization should properly solve two major difficulties (Zienkiewicz et al. 1995). The first one is referred to the constitutive model, which must be mathematically consistent, i.e. it should be able to describe the behavior of materials so that there is no pathological dependence of the solution with the size of the adopted discretization. This difficulty is normally treated by applying regularization techniques, which introduce an internal characteristic length in the equations. The second difficulty arises because the plastic strains are generally concentrated in a narrow band, which is only a small zone of the structure. The analysis strategy should be able to detect and model this small zone, keeping the complexity of the discretization at an appropriate level according to the available computational capacity. The finite element models use several procedures to analyze strain localization problems without introducing excessive complexities in the mesh. Such procedures include the use of shape functions capable to represent a discontinuity in the displacement field, and the automatic generation of adaptive meshes, which capture the discontinuities through automatic refinement of the discretization in the area of interest.

In the dynamic analysis of finite element models of earth dams, it does not seem practical to solve the numerical problems arising from the strain localization phenomenon, by using adaptive meshes coupled with some regularization procedure. This is because external loads change at every moment along the base acceleration history, imposing variable conditions of strain localization to the soil mass. In other words, during the movement a number of failure surfaces may appear in downstream or upstream slopes. These surfaces may be active or not, depending on the evolution of accelerations imposed by the earthquake. On the other hand the width of the zone of localized plastic strains, or failure surface, is in the order of a few nominal diameters of soil grains, while the requirement of a reasonable computation time for the calculations, impose a minimum size of the mesh in the order of 1 to 2 m. In this context some regularization techniques which yield objective results by using either non-local constitutive equations or constitutive equations that take into account the strain gradient, seem inappropriate because they are unable to capture the failure surface, with the mesh sizes used in practice. The possibilities for solving this problem are thus reduced to use constitutive equations that render the softening strain response as dependent on the size of the element, or to introduce discontinuities in the displacement field combined with discrete stress-displacement constitutive equations.

In this paper the material point method is applied to analyze the security of a concrete faced gravel dam, in the search for a method capable of regularizing the strain localization problem, yielding results which are insensitive to the grid layout and, at the same time, solving in a simple way finite deformation problems.

2. THE MATERIAL POINT METHOD

The material point method has evolved from a "particle-in-cell" method for fluid dynamics problems called Fluid Implicit Particle (Sulsky et al., 1995). Implementation of the MPM is easy because several of its fundamental assumptions and mathematical technologies are similar to those used in the widely extended finite element method. The MPM is suitable to model phenomena involving large displacements. Also, it can handle contact/separation problems “naturally”, i.e. there is no need for a separate contact detection algorithm. MPM has been exhaustively applied to problems of impact, penetration, and other phenomena of contact between bodies, transmission of shock waves and analysis of high frequency vibrations.

The material point method represents the material contained in a region as a collection of unconnected particles. An initial mass is assigned to each particle. Particle masses remain fixed throughout the calculation process, thus ensuring global mass conservation. Other initial quantities, such as velocities, strains and stresses, are also assigned to the material points.

The discrete motion equations are not solved at the material points. Instead a support mesh, built to cover the full domain of the problem, is used (figure 1). This mesh is composed of elements of the same type as those used in the finite element method. For the sake of simplicity, it is common to use bilinear regular quadrilateral elements. The variables required to solve the motion equations in the mesh at any step of the analysis are transferred from the particles to the nodes of the mesh by using mapping functions (York et al., 2000). These are the typical shape functions used in the finite element method. The boundary conditions are imposed at the mesh nodes and the motion equations are solved by using an incremental scheme. Then the quantities carried by the material points are updated through the interpolation of the mesh results, using the same shape functions. The information associated to the mesh is not required for the next step of the analysis; therefore it can be discarded provided that the boundary conditions that may have been established are preserved. The discrete form of the governing equations can be found in Sulsky et al, 1995.

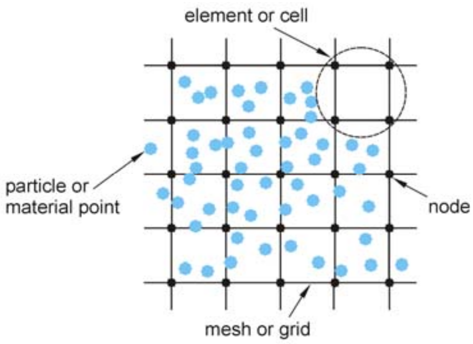


Figure 1. Components of a material point method model

3. PARTICLE MODEL

Figure 2 shows a model of Los Caracoles dam, located in the San Juan Province in central west Argentina. It is a concrete faced dam with a gravel body. The downstream slope, is 1:1.7 (vertical: horizontal) and the upstream slope is 1:1.5.. The particle model shown in figure 2 is made of 31156 particles, a supporting grid with 2m x 2m cell size, and 4 particles in each cell in the initial configuration.

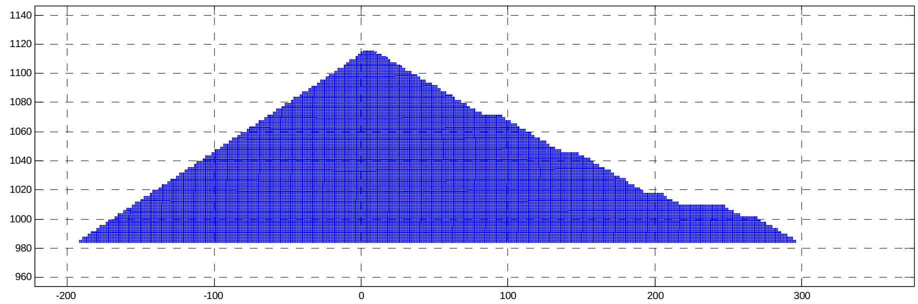


Figure 2. Particle model of a concrete faced gravelly dam (Upstream side left).

4. CONSTITUTIVE MODEL

The material constitutive model used in the analysis is based on the classic theory of plasticity, which defines a yield locus in the stress space. Stress states inside the locus are elastic, while stress states on the locus are plastic. States outside the locus are not possible. The chosen yield surface is a smooth approximation to the Mohr-Coulomb failure surface, proposed by Abbo and Sloan (1995), and defined by Eqns. 4.1. This smooth yield surface avoids the numerical difficulties associated with corners in yield surfaces, while being reasonably close to the original Mohr-Coulomb yield surface (figure 3).

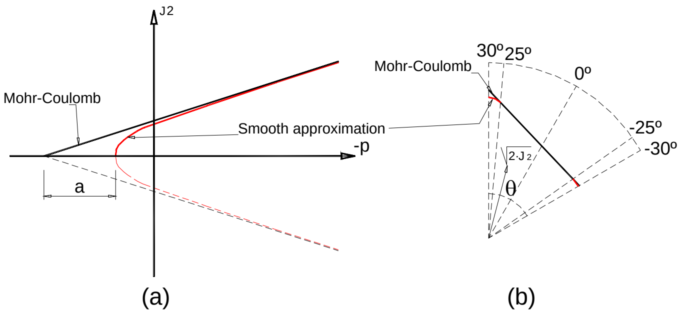


Figure 3. Abbo and Sloan yield surface. (a) Intersection with a constant Lode's angle plane. (b) Intersection with a constant mean pressure plane.

$$F = p \cdot \sin(\phi) + \sqrt{J_2 \cdot K(\theta)^2 + a^2 \cdot (\sin\phi)^2} - c \cdot \cos\phi = 0$$
$$K(\theta) = \begin{cases} A - B \cdot \sin(3 \cdot \theta) & |\theta| > 25^o \\ \cos\theta - \frac{1}{3} \cdot \sin\theta \cdot \sin\phi & |\theta| \le 25^o \end{cases}$$
$$A = 1,4321 + 0,4069 \cdot \text{sign}\,\theta \cdot \sin\phi \tag{4.1}$$
$$B = 0,5443 \cdot \text{sign}\,\theta + 0,6739 \cdot \sin\phi$$
$$a = 0,005 \cdot \cot\phi$$

In Eqn. 4.1 c and ϕ are the cohesion and the internal friction angle. J_2 is the second invariant of the devatoric stress tensor, and θ is Lode's angle. The constitutive model allows strain softening behavior. The post yield friction angle ϕ is evaluated as a decreasing exponential function of the equivalent plastic strain ε_{eq}^{p}:

$$\phi = \phi_{res} + (\phi_{\max} - \phi_{res}) \cdot e^{-\eta \cdot \varepsilon_{eq}^{p}} \tag{4.2}$$

In Eqn. 4.2 ϕ_{res} and $\phi_{\max}$ are the residual and peak friction angle, and η is a parameter of the model. The peak friction angle is assumed to depend on the ratio of the initial confining pressure p' to the atmospheric pressure p_a, according to Eqn. 4.3.

$$\phi_{\max} = 50^o - 2.4 \ln\left(\frac{p'}{p_a}\right) \tag{4.3}$$

Eqn. 4.3 was obtained by fitting experimental data from tests performed on the dam material. A constant Young modulus equal to 500MPa was assumed, and a mass proportional viscous damping, equal to 5% of critical damping for the fundamental period of the model, was used to account for dissipation at low (elastic) strain levels.

The concept of smeared crack or sliding surface in the cell has been applied, i.e. plastic strain concentrated on a sliding surface is distributed throughout the cell. To obtain an objective (non dependent on the size of cells) response in strain softening, the parameter η was calibrated so that for the size of the cell used (2m x 2m), the stress-displacement relation along the sliding surface is close to the softening behavior of the actual material. Figure 4 is a major principal stress vs. displacement plot obtained with $\eta = 100$ and $\phi_{res} = 45^o$ with an initial confining pressure of 400 kPa.

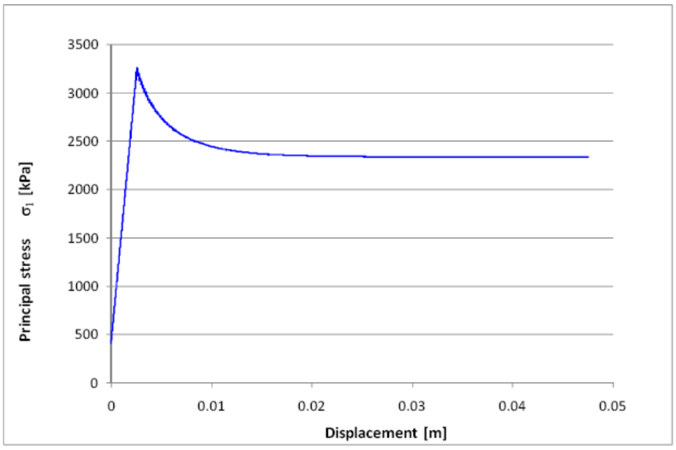


Figure 4. Stress-displacement relation.

5. SELF WEIGHT ANALYSIS

Figures 5 and 6 show the vertical and horizontal effective stress distribution in the body of the dam due to self weight and reservoir water pressure applied on the concrete face (left side of figures). As can be seen there is a lower zone below the concrete face highly compressed due to the action of water pressure. These pressures confine the material, increasing both the stiffness and the strength of the soil, so it is less likely that plastic strains will develop in that area. The downstream slope is far less confined and therefore it is reasonable to expect larger deformations there.

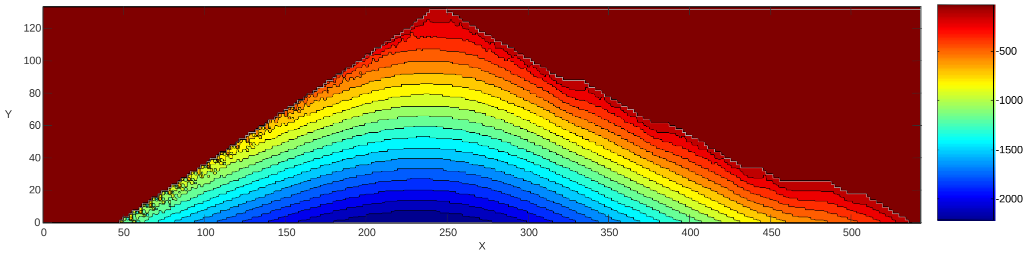


Figure 5.Effective vertical stresses. Self weight and water pressure [kPa].

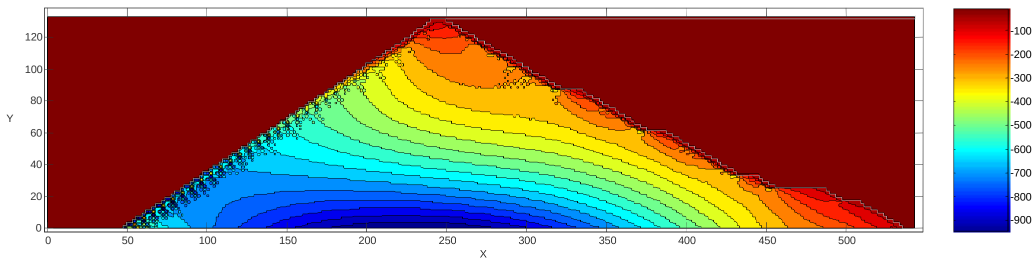


Figure 6. Horizontal stresses. Self weight and water pressure [kPa].

6. DYNAMIC RESPONSE TO A SINE WAVE PULSE

In order to assess the behavior of the model and to detect possible failure mechanisms under dynamic loading, a sine pulse with amplitude equal to 0.3g and period 1.5s was imposed to the rigid base of the model. Figure 7 shows horizontal and vertical displacement histories of the dam crest. Figure 8 shows the distribution of the final equivalent plastic strains in the particles, referring to the original undeformed configuration. This distribution indicates that the model responds with plastic behavior and sliding in the area of low compression of the downstream slope, as it was expected.

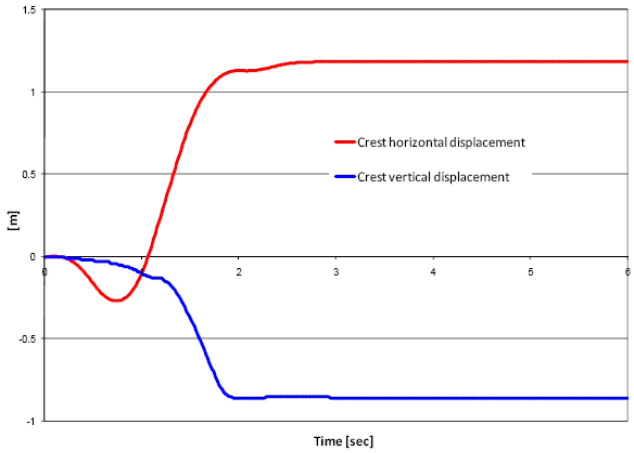


Figure 7. Horizontal and vertical displacement histories of the dam crest. Sine pulse base acceleration.

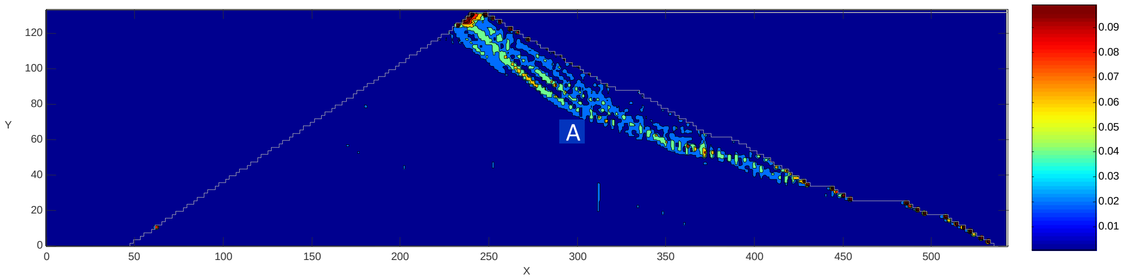


Figure 8. Equivalent plastic strains. Sine pulse base acceleration.

7. COMPARISON WITH NEWMARK'S METHOD

From the acceleration history of point A indicated in figure 8, the displacement of the sliding mass was computed using Newmark's rigid sliding block method (Newmark, 1965). Figure 9 shows the absolute acceleration record of point A, located just under the sliding surface. Figure 10 shows the failure surfaces considered for calculation of the yield acceleration by means of limit equilibrium analysis. Figure 11 compares the displacement estimated by Newmark's method for a 0.22g yield acceleration with the horizontal displacement record of the dam crest calculated using the particle model. The displacements calculated with the particle model are larger than those obtained with Newmark's method, mainly because in the MPM model several "sliding surfaces" are activated. The time evolution of both plots is similar, showing that, in this case, the sliding block analysis yields a qualitative description of the displacement evolution.

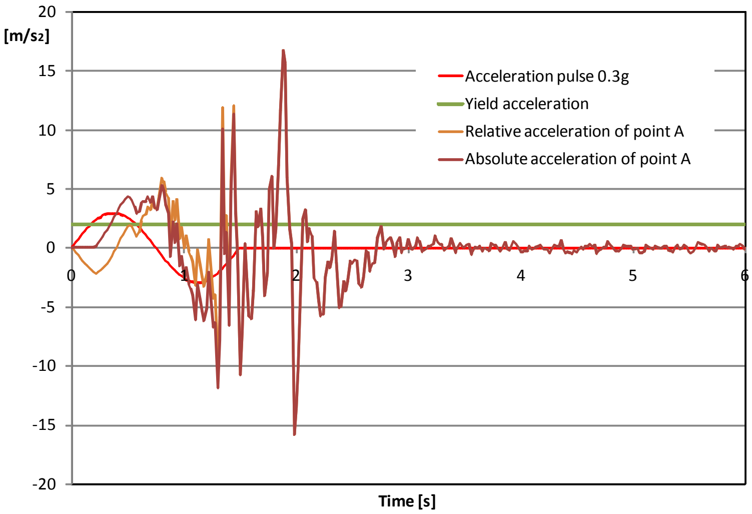
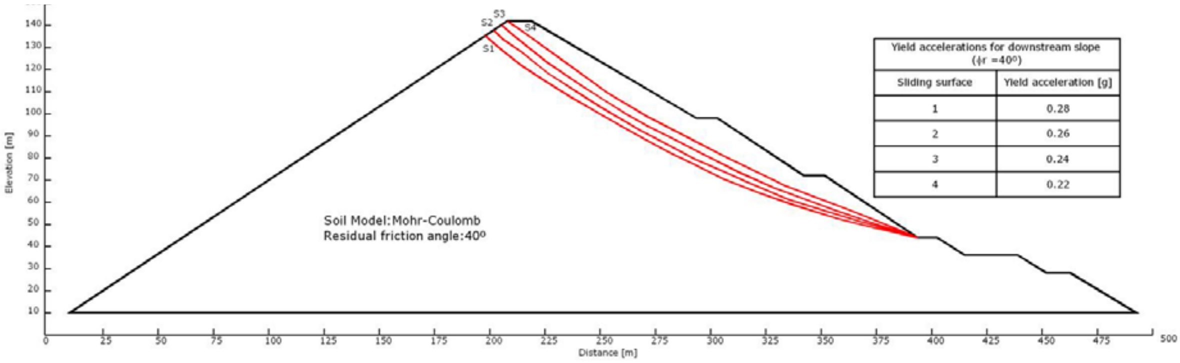


Figure 9. Relative and absolute acceleration of point A.



Yield accelerations for downstream slope (ϕr =40º)	
Sliding surface	Yield acceleration [g]
1	0.28
2	0.26
3	0.24
4	0.22

Figure 10. Yield acceleration for various surfaces, calculated using limit equilibrium.

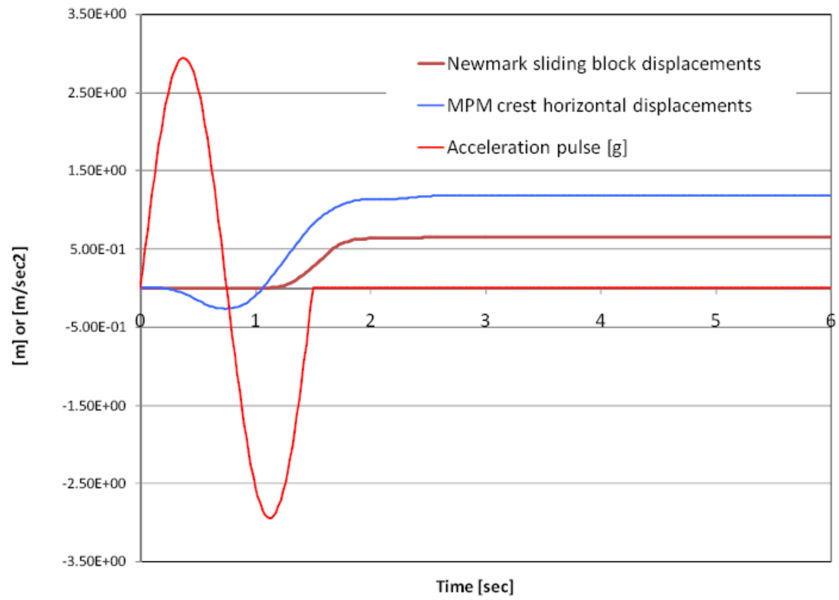


Figure 11. Newmark's method. Crest displacement for a sine pulse acceleration history.

8. DYNAMIC ANALYSIS FOR 1999 CHI-CHI EARTHQUAKE

Figure 13 shows the equivalent plastic strain distribution at the end of a base acceleration history corresponding to the TCU068W record from 1990 Chi-Chi earthquake (figure 12). It is apparent that, as in the previous analysis, a sliding surface develops in the low-compressed downstream slope, and that the largest plastic strains are concentrated in the crest zone. Figure 14 shows the horizontal and vertical displacement record of the dam crest. Notice the large displacement increment coincident with the velocity pulse of the record.

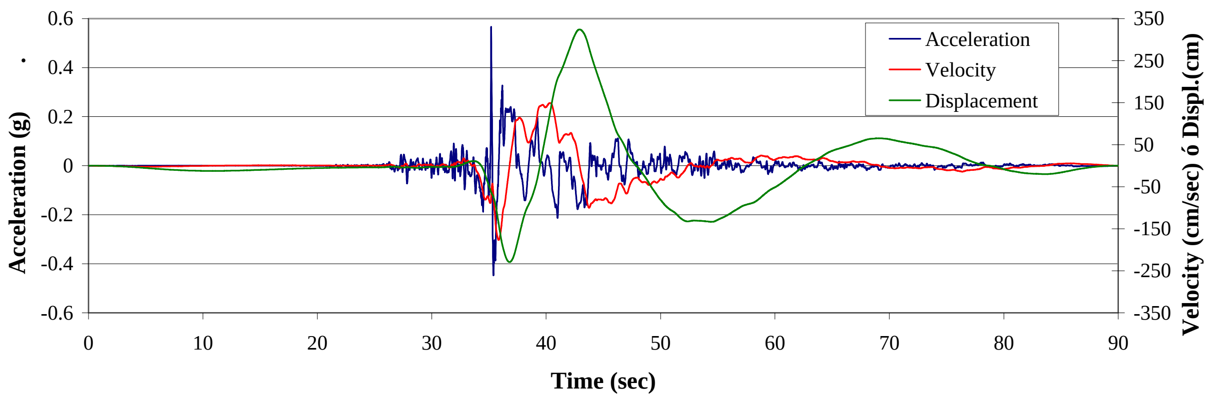


Figure 12. Ground acceleration, velocity and displacement histories for TCU068W record of 09/20/1999 Chi-Chi earthquake.

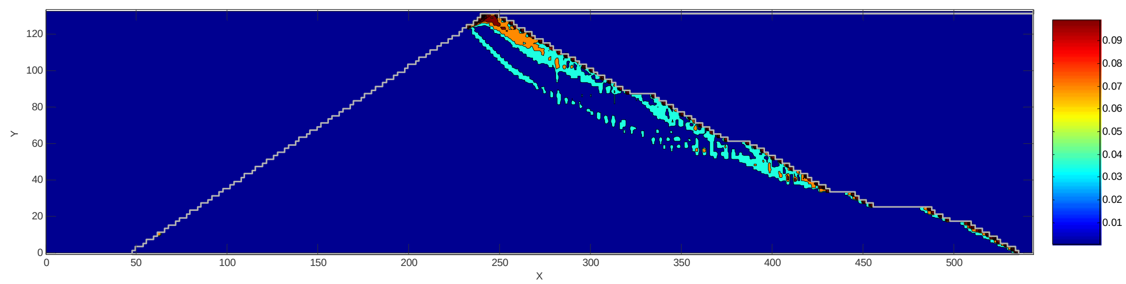


Figure 13. Equivalent plastic strains at the end of TCU068W base acceleration history.

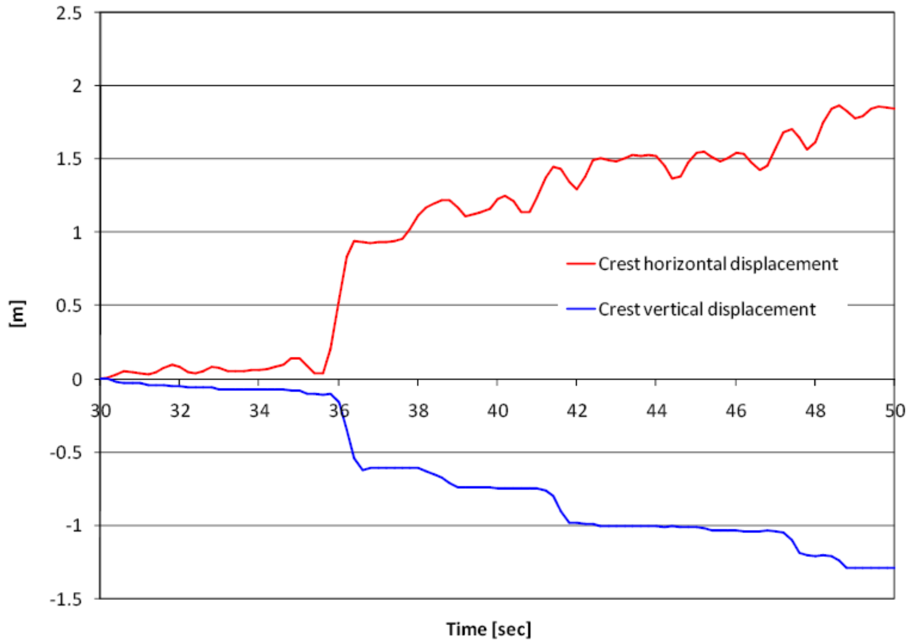


Figure 14. Horizontal and vertical displacements at the dam crest.

9. CONCLUSIONS

The material point method is easy to implement and it allows detection of sliding failure mechanisms. An objective strain softening behavior can be easily obtained by using the concept of smeared crack or sliding surface and calibrating the softening branch of the constitutive equation, in order to get a stress-displacement relation in the sliding surface, which represents the corresponding material relation. The results are not sensitive to the geometry of the supporting mesh since its shape and size is uniform and has no preferred directions. Large displacements are automatically taken into account, by solely using an objective stress rate. The results for the analysis of Los Caracoles dam are consistent with a conceptual model of behavior for a concrete faced earth dam. The results show downstream sliding of the dam crest and the low-compressed area of the downstream slope, without development of plastic strains in the highly compressed area below the concrete face.

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