16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 4159

Registration Code: S-XXXXXXXX

ANALYSIS OF PUNTA NEGRA DAM USING GENERALIZED PLASTICITY MODEL AND THE MATERIAL POINT METHOD

F. Zabala[(1)], R. D. Rodari [(2)], and L. O. Oldecop [(3)]

[(1)] *Director, Earthquake Engineering Research Institute UNSJ, Argentina, fzabala@unsj.edu.ar*
[(2)] *Assistant Professor, Earthquake Engineering Research Institute UNSJ, Argentina, rodari@unsj.edu.ar*
[(3)] *Assistant Professor, Earthquake Engineering Research Institute UNSJ, Argentina, oldecop@unsj.edu.ar*

Abstract

Punta Negra Dam is a 118m high concrete faced gravel dam which is almost finished in San Juan Province, Argentina. This dam is located in the most seismic active zone of Argentina with a very high seismic hazard. Seismic safety of earth or rockfill dams and embankments is strongly conditioned by permanent displacements caused by earthquakes. Also the seismic behavior of concrete faced dams has uncertainties related to, for example, the interaction of the concrete slabs with the dam body and the amplification response of the crest. 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 the soil mass along one or more failure surfaces. 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 change of configuration 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 three dimensional dynamic response of Punta Negra dam.

Keywords: Material Point Method, plastic strain localization, earth dams

1. Introduction

Most of the population of the central-west part of Argentina is concentrated in oases served by artificial irrigation. Water supply to these oases and the need of power generation have encouraged the construction of large dams located upstream of important cities and crop fields. Collapse of one of these works would threat human lives, properties and lifelines. This region also has the greatest seismic activity in the country which is comparable to that of the most seismically active areas in the world.

During the 90's decade of the twentieth century, the knowledge about the seismic motion in near source areas was substantially improved. This was due to the increase in extension and density of strong motion seismographs networks, which were installed in many countries across the world since the 80's. The major increase in the amount of field data and the possibility of obtaining measurements of the motion within the near source area (less than 20 km from the seismic source), have led to the identification of new features of seismic motions that were previously unknown.

Several conclusions can be drawn from the strong motion information that was obtained. First, instrumental information shows that peak ground accelerations occurring in epicentral areas are substantially larger than the limits assumed as physically possible before the 90's decade. In rock sites, the largest ground acceleration for any earthquake magnitude was supposed to be less than 0.8g. This limit has been largely exceeded in many near source records. Also it has been noted that many records contain large velocity pulses. These velocity pulses are known to be caused by directivity (Somerville [1]) and fling effects (Steward [2]). Such features were detected in various earthquakes as Loma Prieta (USA, 1989), Landers (USA, 1993), Kocaeli (Turkey, 1999), Chi-Chi (Taiwan, 1999) and Ducze (Turkey, 1999). These velocity pulses imply the content of long period components in the acceleration record. These long period components particularly affect large earth dams, which have typical natural periods (in the inelastic range) between 0.5 and 2 seconds.

On the other hand, advances in seismic source characterization techniques, fundamentally neotectonic and paleoseismology, have led to a re-evaluation of seismic threat in dams sites, allowing detection of cases in which this threat had been underestimated at the time of dam design. It is important to emphasize the main role that paleoseismicity research plays in the seismic characterization of a region. This research is of main concern for the seismic design of dams, since seismic motions usually specified for the design of these structures have return periods between 5000 and 10000 years, whereas historical seismic activity in the midwest region of Argentina has been observed and recorded just for little more than 200 years. Paleoseismicity research allows the extension of the observation window to the whole Quaternary (1.6 million years), bringing great enhancement to our ability to understand and characterize a region's seismic activity.

2. Seismic verification of dams

According to international practice, dams should be verified under two scenarios: normal operation earthquake and safety earthquake. Under the normal operation earthquake, the dam is expected to sustain seismic action with minor damage that not implies interruption in operation. For the safety earthquake, the structure is expected neither to collapse nor reach a situation of uncontrolled water release, although it is acceptable the occurrence of important damage leading to the need of stopping the operation or even to empty the reservoir in order to carry out reparations.

Analysis of the behavior of a dam sustaining normal operation earthquake does not pose great difficulty in most cases. The design process normally involves stability check, permanent deformation and displacements estimation and stress check in concrete structures. In contrast, when considering the safety earthquake, dams located in areas of high seismic hazard are expected to sustain major damage, taking the structure to a near-collapse stage. In fact, evaluation of safety should include analysis of all possible collapse mechanisms for the structure, in order to study the structure's safety margin with respect to each of these mechanisms. An earthquake can induce over earth dams (Seed [3]) different natures of damage such as: settlement (loss of freeboard), embankments sliding, sliding of dam over foundation and cracking of watertight members and uncontrolled water leaks.

Safety evaluation at high damage stages such as those expected to be produced by the safety earthquake may require sophisticated analysis tools. The analysis should take into account a series of complex phenomena which may include the occurrence of plastic strains, liquefaction or cyclic mobility of saturated granular materials, strain localization in sliding surfaces, cracking, large displacements problems and water seepage with particle erosion.

In general, the kind of analysis tools that would be required to perform a complete analysis of the dam behavior under the action of the safety earthquake are still under development, some of them being used for research purposes, but they are not of common use in engineering practice. The main obstacle to achieve the spreading of sophisticated analysis tools to practice is that they are not yet conveniently tested against measurements and observations of real cases, being an additional difficulty the low number of dams that had sustained intense, epicentral area seismic motion.

3. Analysis tools for evaluation of dams seismic behavior

The level of seismic excitation currently considered for safety assessment of dams takes the analyses into behavior stages that were not considered previously, changing in many cases the scope and methodology of verification. As an example, in the case of earth dams with central clay core, years ago the goal of verification was to ensure that, within the body and the foundation of the dam, the water pressure built-up due to earthquake action is limited to moderate values. This was the logical design criterion following the Terzaghi's effective stress principle, since if the pore pressure increase becomes equal to the existing effective stress, the material's strength would drop to zero and such a dangerous situation should be avoided. This situation does not really imply the structure to be at risk of collapse. Since dense granular materials tend to dilate when subjected to shear strains, the pore pressure would drop immediately if any minor sliding takes place. The undrained residual strength of dense granular materials is high, and therefore the stability of many structures is ensured even if the earthquake does cause high pore pressures. In fact, the dilatant material can reach cyclic mobility during the seismic motion, which implies a momentary loss of stiffness without loss of strength. If this were the case, it would be necessary to estimate permanent displacements caused by the earthquake, and to check that these deformations do not represent a risk to the structure (overtopping risk, for example).

Traditional tools used to perform safety analysis of dams subjected to seismic action are stability evaluation by limit equilibrium methods, dynamic response analysis by means of elastic or linear equivalent finite element models and estimation of permanent displacements using Newmark [4] method.

Seismic safety of earth and rockfill dams is strongly dependent on the magnitude of the final displacements of the dam's body and its foundation after a destructive earthquake. Permanent displacements are caused by volumetric and shear plastic strains distributed within the structure combined with displacements caused by sliding along 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.

In nonlinear 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 using adaptive meshes coupled with some regularization procedure or discrete sliding surfaces that propagate within the dam model. 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 particles, while a reasonable computation time for the calculations requires a minimum size of the mesh in the order of 1 to 2 m. Therefore the practical possibilities to solve this problem are reduced to the use of 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. Non-local constitutive equations or constitutive equations that take into account the strain gradient are unable to capture the failure surface, with the mesh sizes used in practice, but can provide mesh independence of the solution.

Also it is worth noting that the behavior of each type of earthdam structure is different. In zoned dams, upstream displacements of the upstream shell are expected due to the phenomenon of cyclic mobility taking place in the saturated materials. On the contrary, in concrete faced dams, larger permanent displacements are expected to occur in the downstream slope when the reservoir is full. The upstream portion of the dam is under high confinement stresses due to the action of water loads over the concrete face and hence this portion is much stiffer than the rest.

High accelerations are to be expected at the crest of dams, due to dynamic amplification effects. These accelerations may induce sliding failures in the crest area. Other failure surfaces may occur by the presence of weak zones. In these cases, unless special techniques are applied, usual finite element codes cannot satisfactorily reproduce this kind of failure involving strain localization.

4. Material point method

The method was initially described by Sulsky et al. ([5], [6]) and by Sulsky and Schreyer [7]. The material point method represents the material contained in a region as a collection of unconnected material points or lagrangian "particles". An initial mass is assigned to each particle. Particle masses remain fixed throughout the calculation process, thus insuring 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 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. 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. This avoids mesh distortion for large displacements and convection errors. The method is well suited for dynamic problems with large displacements and incorporates in a natural way a non-slip contact algorithm. MPM is similar to the finite element method because the weighting functions which are used in the mesh are of the same type as those used in FEM, and the "particles" could be considered in some cases, as integration points that are not necessarily located at the coordinates of the Gaussian integration points used in FEM. Implementation of the MPM is easy because several of its fundamental assumptions and mathematical technologies are similar to those that support the widely extended finite element method.

MPM is now being used in geotechnical engineering and some interesting applications have been developed. It was applied to the modelling of anchors placed in soil [8], to excavator bucket filling [9], to problems of granular flow in a silo [10], to the simulation of experiments related to fault induced ground deformations [11], to run-out analysis of earthquake-induced soil flows [12] and to geomembrane response to settlement in landfills [13]. Also, a quasi-static version of MPM has been developed for large deformations in geomechanics [14].

Particle models can deal with large displacement without mesh distortion that arises with finite element method. Mesh preparation and 3d models are simpler to develop.

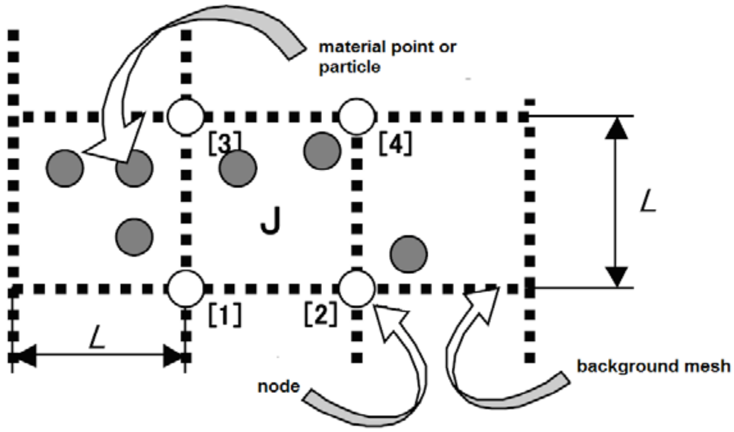

Fig. 1- Components of a material point method model.

5. Punta Negra dam seismic hazard

Punta Negra is a concrete faced gravel dam under construction near San Juan city in Argentina. Figure 2 shows the Uniform Hazard Spectrum for a return period of 10000 years as a result of a probabilistic seismic hazard analysis (PSHA) for the site of Punta Negra Dam [15]. Figure 2 also shows the response spectra obtained from three horizontal strong motion records used for the dam seismic assessment.

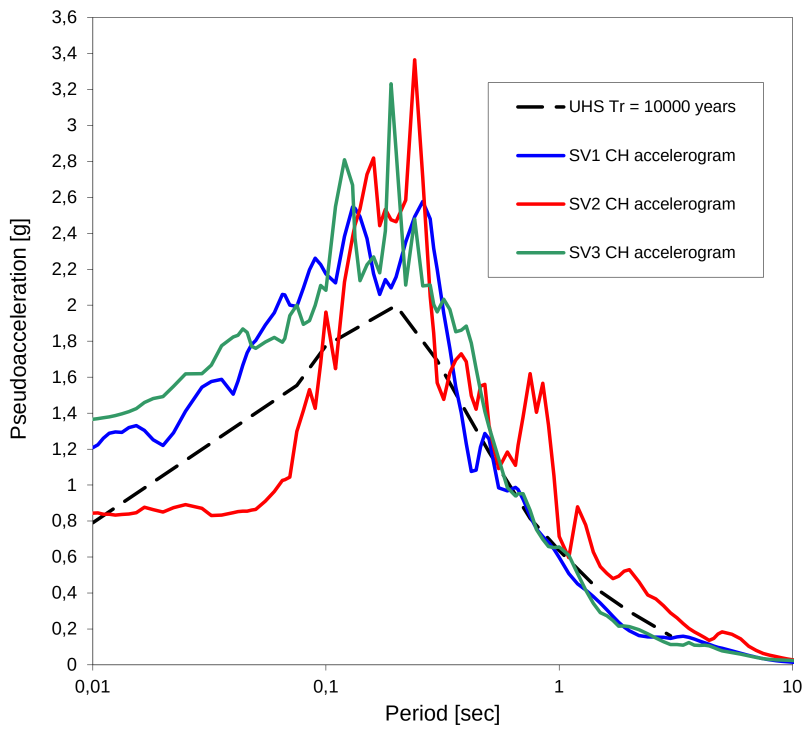

Fig.2- Uniform Hazard Spectra for Punta Negra dam

5. Analysis of Punta Negra dam

Figure 3 shows a simplified three-dimensional particle MPM model of Punta Negra dam. The initial downstream slope is 1:1.65 (vertical: horizontal) and the upstream slope is 1:1.5. With the aim of exploring the capabilities of MPM in dam analysis a first coarse model was developed with 56250 particles, and a supporting grid of 5m x 5m cell size with 1 particle per cell in the initial configuration. The concrete face was not modelled. A solid obtained from a 3d Delaunay triangulation of the initial particles position using Paraview [16] is shown in Figure 4.

The results presented here were computed using a modified version of Pastor–Zienkiewicz constitutive model [17] which is described later.

Figure 5 shows the dam body clipped by a symmetry plane (upstream-downstream). In this figure particle colors refer to vertical effective stresses due to self-weight and reservoir water pressure applied on the concrete face (left side of figure). As can be seen there is a zone below the concrete face highly compressed due to 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.

Shear equivalent plastic strains (equation 1) and displacements of the particles are shown in Figure 6 for a step during the model dynamical analysis with SV2 accelerogram and empty reservoir. Figure 7 shows a 2D cross-section derived from these results. In this case the larger shear strains are developed in the more stepped upstream slope.

$$\dot{\varepsilon}^{p}_{eq} = \sqrt{\frac{2}{3}\dot{e}^{p}_{ij}\dot{e}^{p}_{ij}} \qquad (1)$$

$\dot{\varepsilon}^{p}_{eq}$: equivalent plastic shear strain increment,

$\dot{e}^{p}$: : deviatoric plastic strain

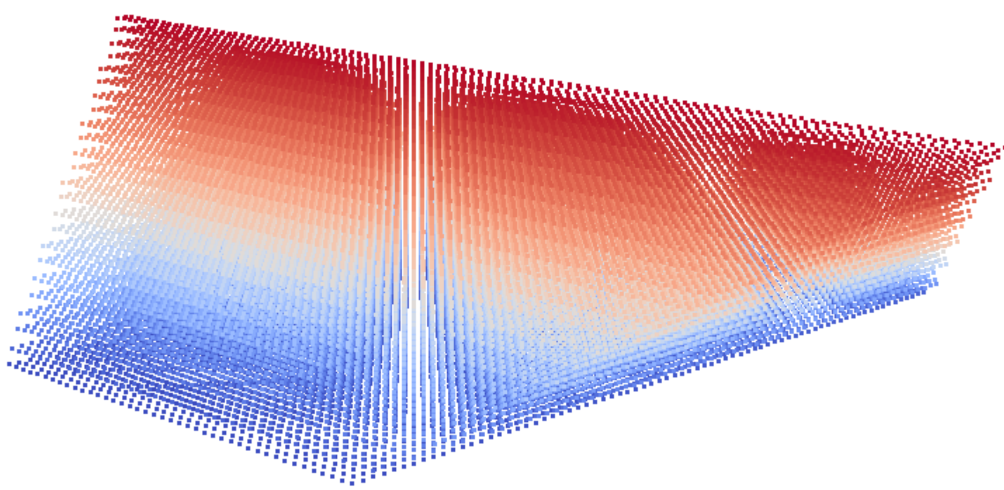

Fig. 3- CFRD tridimensional particle model. Initial particle position

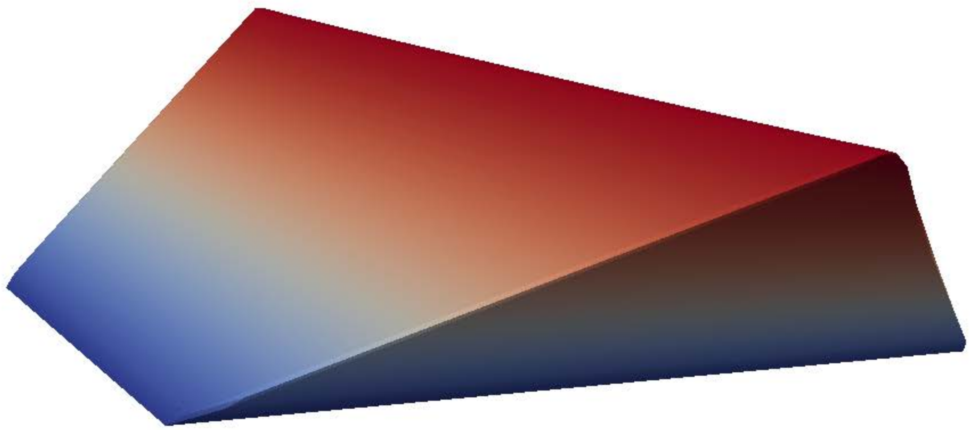

Fig.4 - 3D Delaunay triangulation of the initial particles position

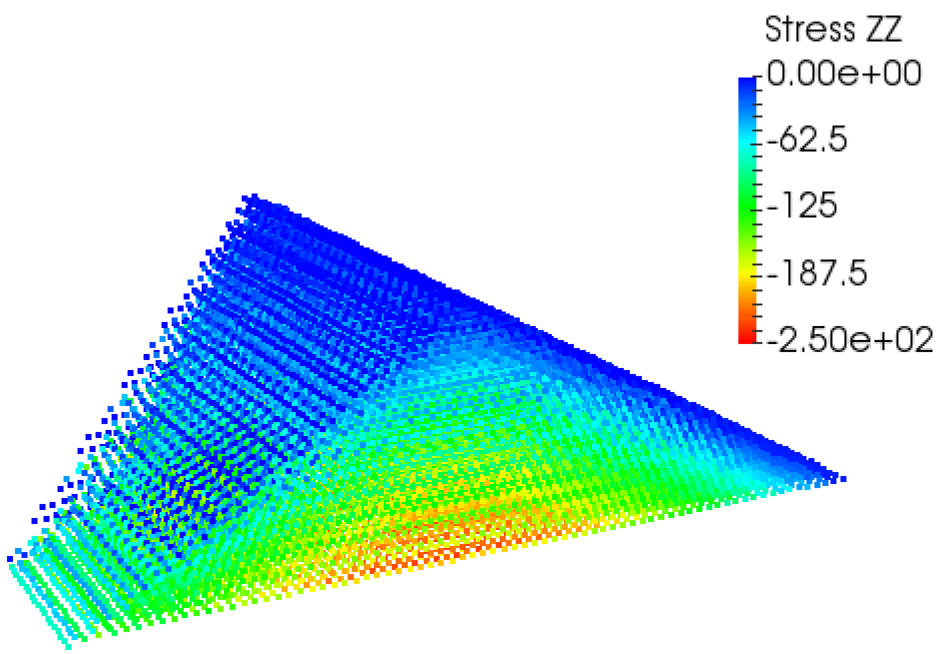

Fig.5 - Particle model central cross section. Vertical stresses [t/m^2]

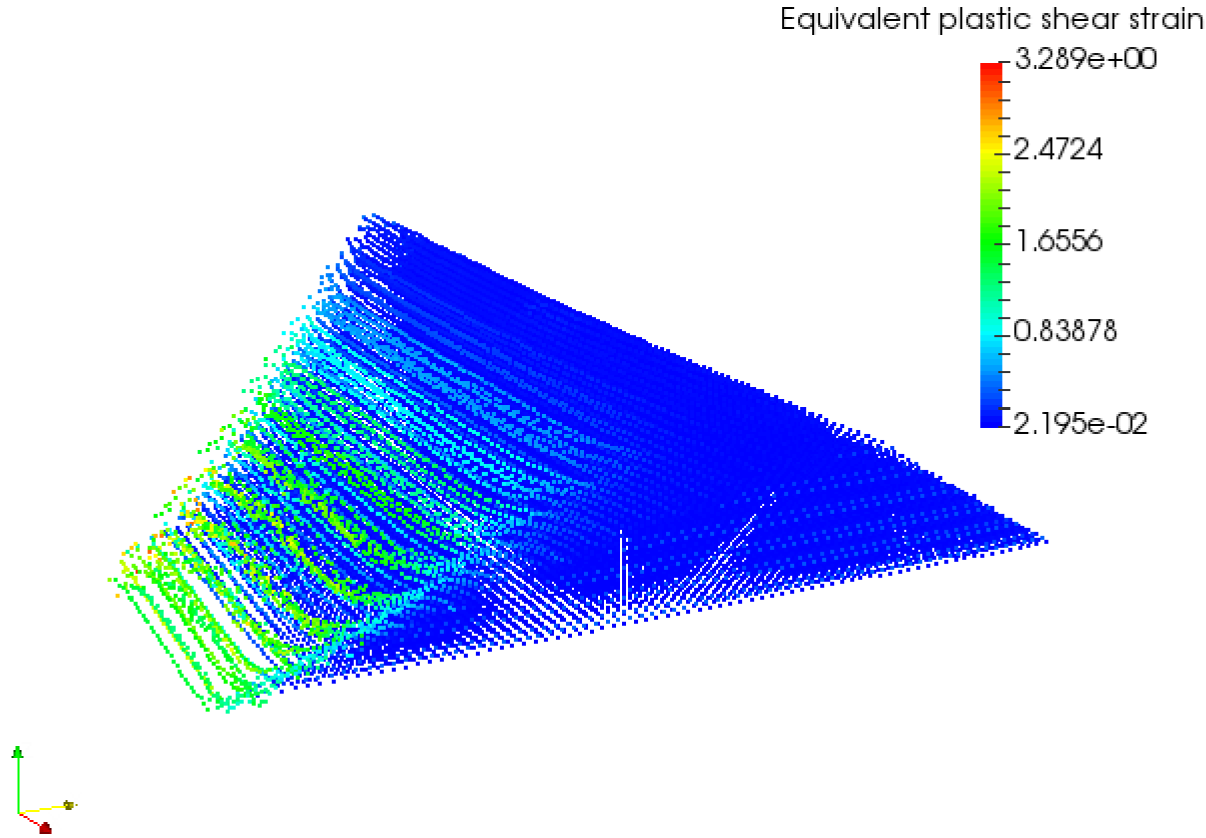

Fig.6 - Particle model. Displacements and equivalent plastic shear strain

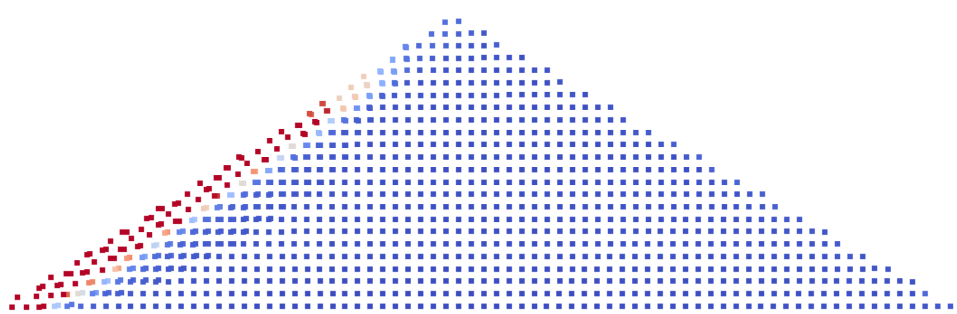

Fig. 7- Dam central cross section of particle model. Equivalent plastic shear strain

5.1 Generalized plasticity constitutive model

Figure 3 shows Pastor – Zienkiewicz model [17] for sands was used for the analysis. This model is formulated in the frame of generalised plasticity Constitutive matrix is defined as:

$$D^{ep} = D^e - \frac{D^e n_g n_f^T D^e}{n_g D^e n_f + H_{L/U}} \qquad (2)$$

The plastic modulus $H_{L/U}$ is a hardening parameter that is a function of effective stress ratio, effective pressure and accumulated strain. The model is formulated in terms of the three invariant: effective mean pressure p', deviatoric stress q, and Lode angle. The flow direction is a function of dilatance d, for loading and unloading.

$$n_g = (n_v, n_s, n_\theta) \qquad \mathrm{d} = (1+\alpha)(\mathrm{Mg} - \eta)$$

$\eta = q / p'$: stress ratio

α = material constant

M_g = slope of the critical state line

$$n_v = \frac{d}{\sqrt{(1+d^2)}} \qquad n_s = \frac{1}{\sqrt{(1+d^2)}}$$

The loading direction vector is defined in a similar form:

$$n_{L/U} = (n_p, n_q, n_\theta) \qquad \mathrm{d_f} = (1+\alpha)(\mathrm{Mf} - \eta)$$

$$n_q = \frac{1}{\sqrt{(1+d_f^2)}} \qquad n_p = \frac{d_f}{\sqrt{(1+d_f^2)}}$$

and the plastic modulus H for loading is:

$$H_L = H_0 p'(H_v + H_s) H_f H_{DM} \tag{3}$$

$$H_v = 1 - \frac{\eta}{M_g} \qquad H_s = \beta_o \cdot e^{(-\beta_1 \cdot \xi)}$$

The details of the model formulation can be found in [17].

5.2 Modification of P-Z constitutive model.

The model was modified due to the difficulties founded on selecting appropriate parameters that can represent accurately the material steady state strength, using a plastic modulus for loading:

$$H_L = H_0 p'(H_v + H_s H_{ff}) H_f H_{DM} \tag{4}$$

$$H_{ff} = 1 - \frac{\eta}{\eta_{CS}} \qquad \eta_{CS} = \eta_f \left[1 - \left(\frac{\mathrm{p'}}{\mathrm{p_{cr}}} \right)^{\alpha} \left(1 - \frac{M_g}{\eta_f} \right) \right]$$

$$\eta_f = (1 + 1/\alpha)\, \mathrm{M_f}$$

η_{CS} : limit stress ratio that causes $H_{ff} = 0$ for $p' = p'_{cr}$ and $\eta = M_g = \eta_{CS}$

p'_{cr} is the effective pressure which the material reach under steady state conditions and depends on the void ratio. (Figure 8).

5.2 Strain localization

In order to take account of strain localization 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 (Figure 9). To obtain an objective response (nondependent on the size of cells) in strain softening we follow the approach described in reference [18]. The volume averaged total strain rate of particles can be expressed as:

$$\dot{\boldsymbol{\varepsilon}} = f\dot{\boldsymbol{\varepsilon}}_i + (1-f)\dot{\boldsymbol{\varepsilon}}_o \qquad (5)$$

Where subscripts ''i'' and ''o'' are used for the strain inside and outside the localization region, respectively and f = h/L is the volume fraction of the localization band to the total particle volume.

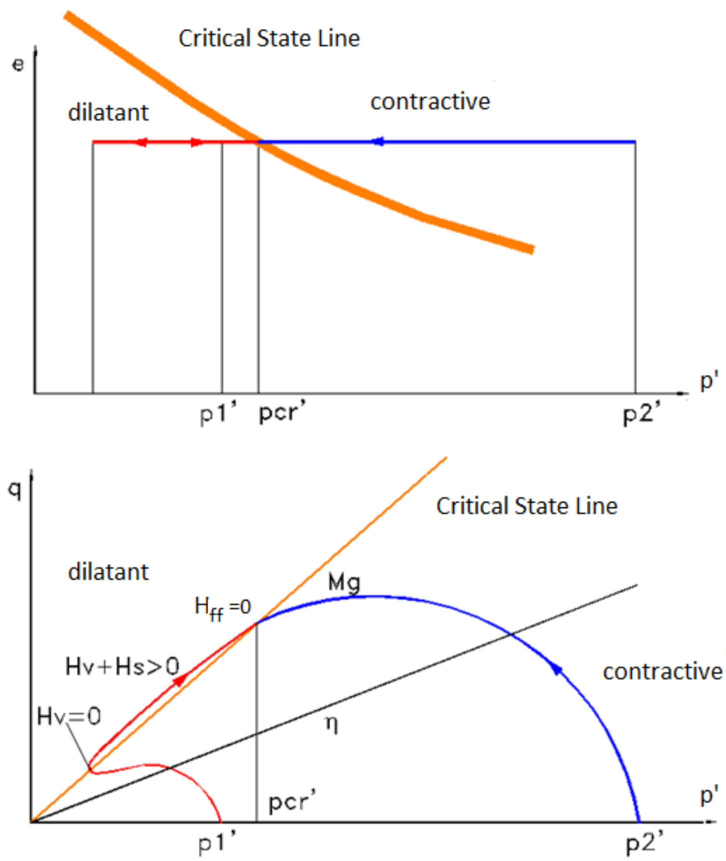

Fig.8- Modification of P-Z constitutive model

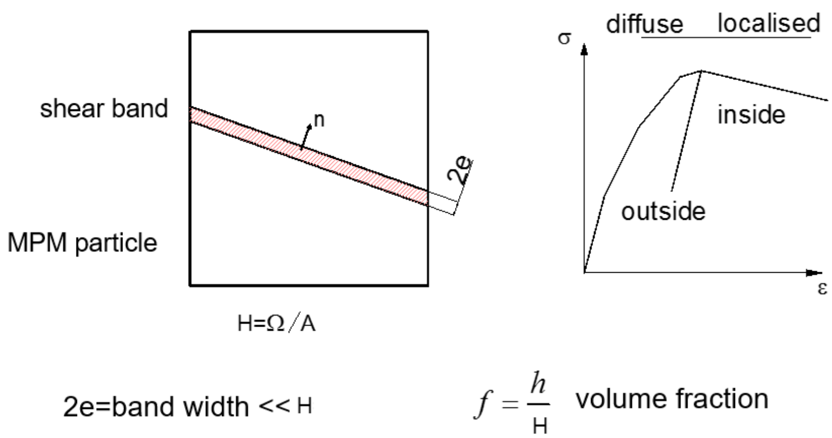

Fig.9- Smeared crack modelling of shear band

6. Conclusions

The Material Point Method can help to improve the dynamic analysis of geotechnical structures taking into account in a simple manner many of the issues related. The model presented in this paper captures the principal features of the expected dam behavior and can deal with large displacements without mesh distortion. The coarse 3D mesh used cannot represent accurately the geometry of localized shear surfaces, and also the concrete face is not modelled with particles. Future efforts should be directed to model the interaction of the concrete face with the supporting rockfill and obtain a proper definition of localization surfaces.

7. References

[1] Somerville, P.G., N.F. Smith, R.W. Graves, and N.A. Abrahamson (1997). Modification of empirical strong ground motion attenuation relations to include the amplitude and duration effects of rupture directivity. *Seismological Research Letters* **68**, 199-222.

[2] Steward, J.P., S.J. Chiou, J.D. Bray, R.W. Graves, P.G. Somerville y N.A. Abrahamson (2001). Ground Motion Evaluation Procedures for Performance-Based Design. PEER Report 2001/9. *Pacific Earthquake Engineering Research Center, Berkeley*.

[3] Seed HB. (1981). Earthquake-resistant design of earth dams *International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics* Vol III, St. Louis, Missouri.

[4] Newmark N.(1965). Effects of earthquakes on dams and embankments. *Geotechnique*; **15**(2): 139-160.

[5] Sulsky, D., Z. Chen, & H. L. Schreyer (1994). A Particle Method for History-Dependent Materials. *Computer Methods in Applied Mechanics and Engineering*. **118**(1), 179-96.

[6] Sulsky D, Schreyer HL, Zhou S-J. (1995).Application of a particle-in-cell method to solid mechanics. *Computer Physics Communications*. **87**: 236-252.

[7] Sulsky, D. & H. L. Schreyer (1996). Axisymmetric Form of the Material Point Method with Applications to Upsetting and Taylor Impact Problems. *Computer Methods in Applied Mechanics and Engineering*. **139**(1), 409-29.

[8] Coetzee, C. J., Vermeer, P. A & Basson A. H. (2005). The Modelling of Anchors using the Material Point Method. *International Journal for Numerical and Analytical Methods in Geomechanics*. **29**(9), 879-895.

[9] Coetzee, C. J., Basson A. H., &. Vermeer P. A. (2007) Discrete and Continuum Modelling of Excavator Bucket Filling. *Journal of Terramechanics*. **44**(2), 177-186.

[10] Więckowski Z., Youn S.K., Yeon Y.H. (1999).A particle-in-cell solution to the silo discharging problem. *Int. J. Numer. Meth. Engng*.. **45**, 1203-1225.

[11] Johansson, J. & K. Konagai (2007). Fault Induced Permanent Ground Deformations: Experimental Verification of Wet and Dry Soil, Numerical Findings' Relation to Field Observations of Tunnel Damage and Implications for Design. *Soil Dynamics and Earthquake Engineering*. **27**(10), 938-956.

[12] Konagai, K., Johansson J. & Itoh H. (2004) Pseudo-three dimensional lagrangian particle finite difference method for modeling earthquake induced soil flows. *13th World Conference on Earthquake Engineering*. No 547,

[13] Zhou, S., Stormont J. & Chen Z. (1999).Simulation of Geomembrane Response to Settlement in Landfills by using the Material Point Method. *International Journal for Numerical and Analytical Methods in Geomechanics.* **23**(15), 1977-1994

[14] Vermeer P. A.,Beuth L. & Benz T. A. (2008) Quasi-Static Method for Large Deformation Problems in Geomechanics *The 12th International Conference of International Association for Computer Methods and Advances in Geomechanics* (IACMAG) Goa, India,

[15] Oldecop L & Zabala F. (2009) Evaluación de la amenaza sísmica para el proyecto hidroeléctrico Punta Negra. *Instituto de Investigaciones Antisísmicas. Universidad Nacional de San Juan,*

[16] The ParaView Guide: A Parallel Visualization Application by Utkarsh Ayachit. (2015).Published by Kitware Inc. in, Clifton Park, NY.

[17] Pastor M. Zienckiewics,O.C. and Chan A.H. (1990).Generalized plasticity and the modelling of soil behavior. *International Journal for numerical and analytical methods in geomechanics.* **14**.

[18] Nguyen G.D., Korsunsky A,M and Einav I...A (2014). Constitutive modelling framework featuring two scales of behaviour: *Fundamentals and applications to quasi-brittle failure. Engineering Fracture Mechanics* **115** 221–240,.