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INVESTIGATION OF DYNAMIC STABILITY OF DAMS CONSTRUCTED OF LOCAL MATERIALS IN HIGH SEISMICITY REGIONS

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ABSTRACT

Due to the extremely nonlinear character of soil media , they are treated nonlinearly, using the Mohr-Coulomb's criterion regarding the ultimate bearing capacity which is characteristic for the friction material media. The investigations are performed by consideration of effective stresses wherefore the distribution of pore pressure through the coherent media is analyzed previously. The differential equation of motion is solved by using the incremental step-by-step linear acceleration method which incorporates the Wilson-Θ method for direct integration Within each time increment, conducted is the iterative procedure referred to as "Load Transfer Method" which enables balancing of the residual forces and satisfying of the dynamic equilibrium conditions within the considered time increment. A comparison between the obtained nonlinear dynamic response and the linear response of the system is made. The stability of the structure is checked by its both linear and nonlinear treatment.

KEYWORDS

Mohr-Coulomb's criterion, Load Transfer Method, nonlinear analysis, stability, dynamic analysis, earth fill dams

INTRODUCTION

The Mohr's hypothesis for failure and the Coulomb's equation for ultimate bearing capacity incorporated into the Mohr's diagram are known as the Mohr-Coulomb's criterion for ultimate bearing capacity representing an extension of the Tresca's criterion and its adaptation to frictional materials. According to this author, the critical shear stress is a function of the normal stress acting along the normal of the potential octahedral plane, the cohesion and the angle of internal friction. The normal stress in the octahedral plane is the mean stress, which means that the Mohr-Coulomb's criterion for yielding takes into account the spherical component of the stress tensor which is another difference from the Tresca's criterion and the criterion of maximal distorsional energy of von Mises, where yielding is determined only from the deviation component. It is proved that the behavior of soil and its resistance are controlled by the spherical tensor wherefore it must not be neglected. Also, the spherical tensor affects the ductility of the material, i.e., under higher spherical stress, the soil has the capability of developing larger deformations prior to failure.

EXCESSIVE STRESSES AND RESIDUAL FORCES

The stress state within the frameworks of the finite element is expressed through the componental stresses, the principal stresses, the octahedral tangential stress and the mean stress. These stresses and equation $\tau_{gr} = C\cos\varphi + \sigma_m \sin\varphi$ are used to define the ultimate elastic bearing capacity within each finite element. The finite elements for which the defined ultimate shear bearing capacity (τ_{gr}) is less than the manifested octahedral shear stress (τ_{max}) according to the Mohr-Coulomb's criterion, exert a plastic behavior. Depending on the size of the manifested mean stress (σ_m), the following two different states are distinguished:

a) Plastic behavior due to exceedence of the shear stress $\sigma_m < C\cot\varphi$.

b) Occurrence of cracks in the soil medium due to exceedence of the tensile strength $\sigma_m > C\cot\varphi$.

a)The manifested stress state given by the Mohr's circle A Fig. 1. represents a stress state obtained by treating the problem as a linear one. It cannot be sustained by the soil medium at the considered point, since the ultimate bearing capacity given by circle B is less than the manifested one. Therefore, it is necessary to reduce the stress state up to the level of ultimate stresses defined by the Mohr's circle B. These ultimate stresses indicate the beginning of the plastic yielding. Defined are the "excessive" stresses $\{\Delta\sigma\}_e$ and $\{\Delta\tau\}_e$ that have to be eliminated using an iterative procedure .

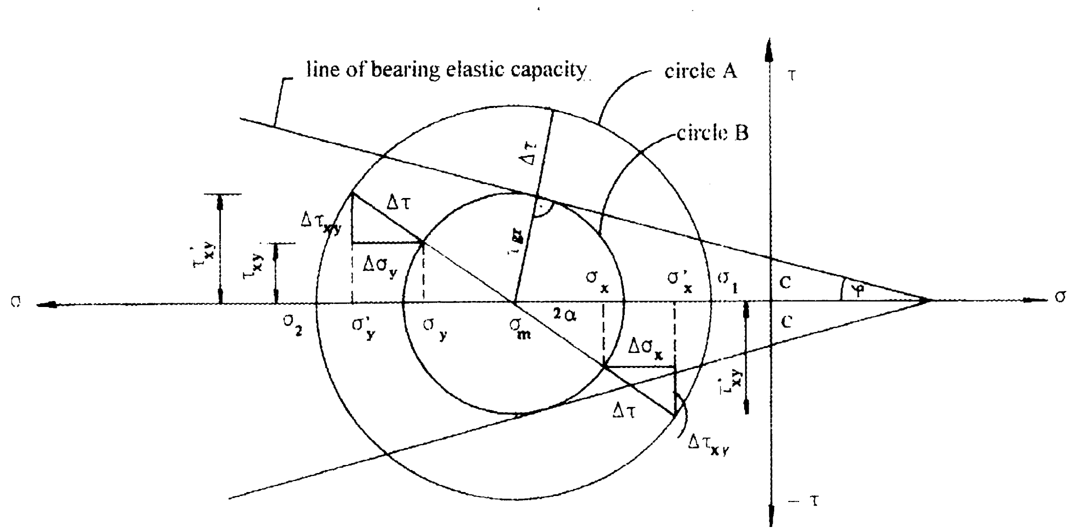


Fig. 1.

$$\{\Delta\sigma\}'_e = \begin{Bmatrix} \Delta\sigma x \\ \Delta\sigma y \\ \Delta\tau xy \end{Bmatrix} = \begin{Bmatrix} -\Delta\tau\cos 2\alpha \\ \Delta\tau\cos 2\alpha \\ -\Delta\tau\cos 2\alpha \end{Bmatrix} \qquad \Delta\tau = \tau\,\max - \tau gr \tag{1}$$

b)In the case the mean stress σ_m is higher than the tensile bearing capacity of soil $C\cot\varphi$ Fig. 2. , which is represented by point P in the σ-τ coordinate system, the Mohr's circle A has to be reduced to the stress state defined by point P. The vector of excessive stresses in this case is the following:

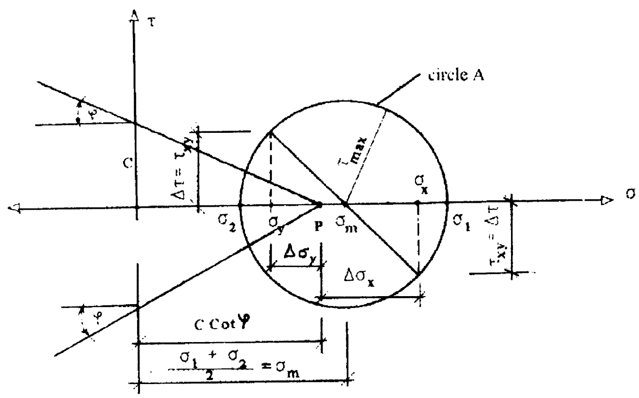


Fig. 2.

$$\{\Delta\sigma\}_e^i = \begin{Bmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau_{xy} \end{Bmatrix} = \begin{Bmatrix} Cctg\varphi - \sigma_x \\ Cctg\varphi - \sigma_y \\ -\tau_{xy} \end{Bmatrix} \qquad \Delta\tau = \tau\ \max - \tau gr \tag{2}$$

The convergence within the frameworks of each finite element and for each integration point is monitored through the norm of excessive stresses which is computed by using the following equation:

$$SNORM = \sqrt{\Delta\sigma_x^2 + \Delta\sigma_y^2 + \Delta\tau_{xy}^2}\ /3 \tag{3}$$

In case of pure tension, during the iterative processes, it often happens that the norm of excessive stresses of the element increases, i.e., the iterative process within the frames of the finite element diverges. The Mohr's circle extends out of the defined range of tensile bearing capacity of soil. These stresses cannot be sustained wherefore cracking of the finite element in the direction normal to the developed planes of maximal tensile stresses takes place. At this moment, the excessive stresses are equal to the developed stress state at the integration point. According to the manifested crack, the stress state of such a finite element equals zero and it is the initial stress state for the next iteration.

The matrix differential equation for the dynamic equilibrium of a discrete system using the substructure concept is the following:

$$M^{**}U + C^{**}U + K^{**}U = P^{**}$$

$$M^{**} = \sum_{i=1}^{n} M^{**} \qquad C^{**} = \sum_{i=1}^{n} C^{**} \qquad K^{**} = \sum_{i=1}^{n} K^{**} \qquad n = 1, iter \tag{4}$$

The matrices indicated by the symbol (*) represent the matrices of the superelements corresponding to the external Guyan's points of the substructures. When assembled, they constitute the global matrix of the system as a whole. The superelement matrices are obtained by Guyan's reduction. The system of equations (4) is solved by applying the direct integration method which enables that the damping matrix be expressed in an explicit form, i.e. the Rayleigh's damping concept is applied. The step-by-step linear acceleration method is used whereby the problem is reduced to solving of successively variable linear systems, whereat if a nonlinear problem is treated, it is possible to correct stiffness and damping matrices at the beginning of each time step. The Wilson-θ method is used in order to provide an unconditional stability of the integration process.

The incremental form of the differential equations of motion is the following:

$$M\hat{\Delta}\ddot{U}+C\hat{\Delta}\dot{U}+K\hat{\Delta}U=\hat{\Delta}P \qquad (5)$$

Symbol ($\wedge$) indicates that the increments refer to a prolonged time interval in accordance with the Wilson-θ method. Once the unknown increments in the prolonged time step are defined in equation (5), using linear interpolation in the normal time step (Δt), the response at the end of each discrete time is obtained by superposition. However, the acceleration at the end of the i-th time step is obtained directly, by solving the differential equation for the dynamic equilibrium at time moment ($t_{i-1}+\Delta t$). In this way, accumulation of error is eliminated in the process of successive repetition of the numerical procedure and due to the approximate character of the method. In addition to the incremental "step-by-step" method, the technique of solving the nonlinear problem is amended by an iterative procedure by means of which balancing of the residual forces is performed within the frames of each step. Such mixed methods are the most appropriate for solving of nonlinear dynamic problems ,since in the incremental method, matrices Ki and Ci are constant within the domain of each increment. The equilibrium condition is achieved with several successive convergent iterations whereat the residual forces of each iteration are taken as initial values in the next iteration. Presented in this paper is the modified Load Transfer Method. The main phases of the Load Transfer Method are the following:

1. Solved within the frames of each iteration is the incremental differential equation for dynamic equilibrium and defined is the increment of the vector of response at the external points of the system. At the end of each iteration, the response is defined by simple summing of the effects, i.e.:

$$\{U_n\}=\{U_o\}+\sum_{i=1}^{n}\{U_i\} \quad \{U_n\}=\{U_o\}+\sum_{i=1}^{n}\{U_i\} \quad \{U_n\}=\{U_c\}+\sum_{i=1}^{n}\{U_i\} \quad n=1,iter \qquad (6)$$

2. Using the displacement vector increment, defined within the frames of each iteration is the vector of increment strains and increment stresses in the finite elements, using the known relationships in the finite element method:

$$\{\Delta\varepsilon\}=[B]\{\Delta U\} \qquad \{\Delta\sigma\}=[D]\{\Delta\varepsilon\} \qquad (7)$$

Here, the vector of incremental displacements {ΔU} for each finite element is obtained from the incremental displacements of the internal and the external nodes of the substructure. The incremental displacements at the internal nodes of the substructure are obtained using the Guyan's transformation. The total vectors of strains and stresses at the end of each iteration are computed as follows:

$$\{\varepsilon_i\}=\{\varepsilon_0\}+\sum_{i=1}^{n}\varepsilon_i \qquad \{\sigma_i\}=\{\sigma_0\}+\sum_{i=1}^{n}\sigma_i \qquad n=1,iter \qquad (8)$$

3. At the end of each iteration, an insight into the stress state of the whole discrete model is obtained. Based on the stress state, selected are those finite elements that are in a state of nonlinearity, i.e., those in which there is exceedence of the ultimate stresses. For these finite elements, the "excessive" stresses are defined as a difference between the manifested and the ultimate stresses that have to be eliminated. For this purpose, the "residual" forces are defined according to the following equation:

$$\{f^{e}_{rez}\}=-\int_{v}[B]^{T}\{\Delta\sigma\}_{e}\,dv \qquad (9)$$

4. Solving again the system of equations under the effect of the residual forces, defined are the vectors of residual displacements and stresses. The vector of residual stresses and displacements is summed up with the vector of stresses and displacements from the previous iteration. Since the residual forces are applied on a linear system with invariable stiffness matrix , once the stress state is defined, it is proved that the excessive

stresses still exist, so that several iterative procedures are needed to reduce the vector of excessive stresses and residual forces to the level of required accuracy.

RESULTS

The prepared software package PROCESS was applied for a rock fill dam with a height of 85 m, excited by the Imperial Valley earthquake with $a_{max\ x}$= 0.36g. From the performed analysis, presented are the time histories deformations and accelerations at a characteristic point of the dam crest Fig. 3. to Fig. 5. and the time histories of stresses at the certain point in the rock fill of the dam Fig. 6 to Fig. 10., snapshot of deformations at time moment t = 2.26 sec for linear and nonlinear analysis Fig. 11. and the location of the developed cracks in time t = 2.205 sec Fig. 12. Computed are the time histories of the safety coefficient against sliding in assumed potential sliding surfaces. Presented is only one time history of the safety coefficient for a characteristic sliding surface at depth of 13.0 m from the crest Fig. 13. The presented figures lead to the conclusion that there exists a considerable difference between the linear and the nonlinear response, particularly during the occurrence of a strong earthquake. Applying this method of analysis, one can obtain information on the zones of the dam body where cracks are to be expected. Assuming a criterion that the safety coefficient ranges between 1 to 1.05, one can estimate the depth (considered from the dam crest) up to which the potential sliding surfaces have a tendency to exert instability.

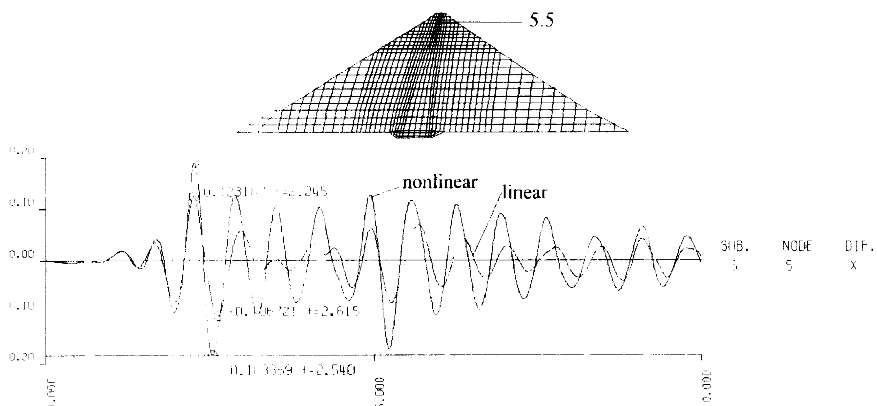


Fig. 3. Time history of relative displacement ts=0. te=10.sec.
Imperial Valley earthq.M=6.5 Ao=3.6 m/s^2 dt=0.005 Comp.1
Imperial Valley earthq.M=6.5 Ao=2.7 m/s^2 dt=0.005 Comp.2

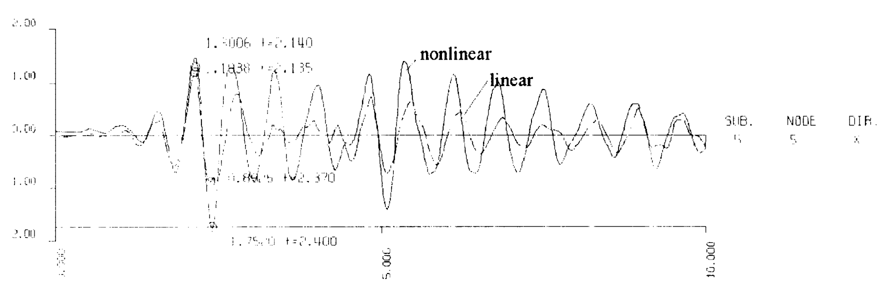


Fig. 4. Time history of relative velocity ts=0. te=10.sec.

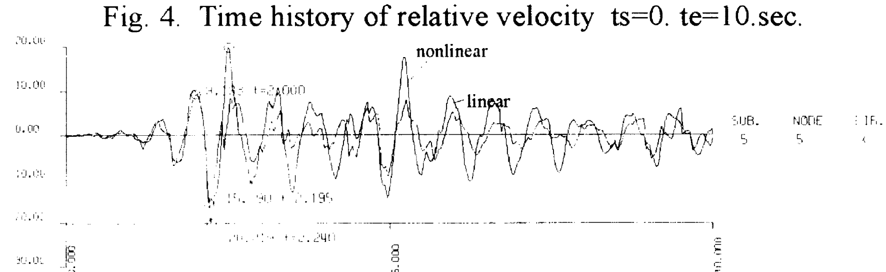
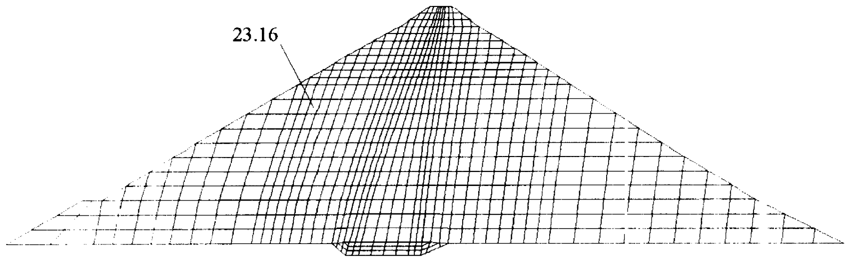


Fig. 5. Time history of relative acceleration ts=0. te=10.sec.



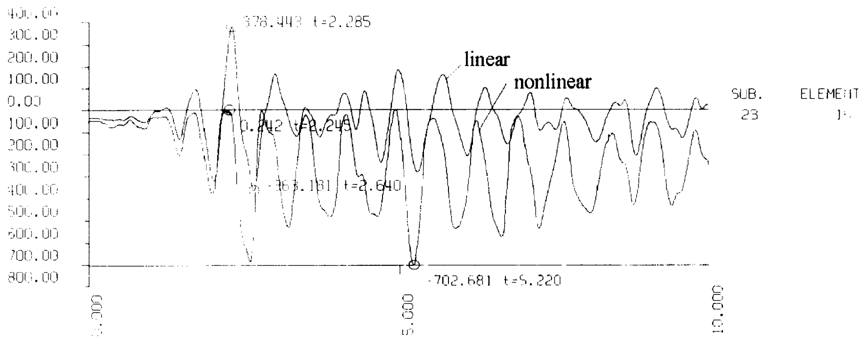


Fig. 6. Time history of componental stresses Gx ts=0. te=10.sec.
Imperail Valley Earthq. M=6.5 Ao=3.6 m/s^2 dt=0.005 Com.1
Imperail Valley Earthq. M=6.5 Ao=2.7 m/s^2 dt=0.005 Com.2

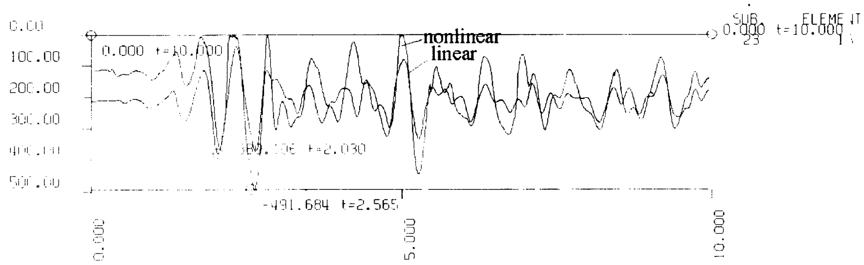


Fig. 7. Time history of componental stresses Gy ts=0. te=10.sec.

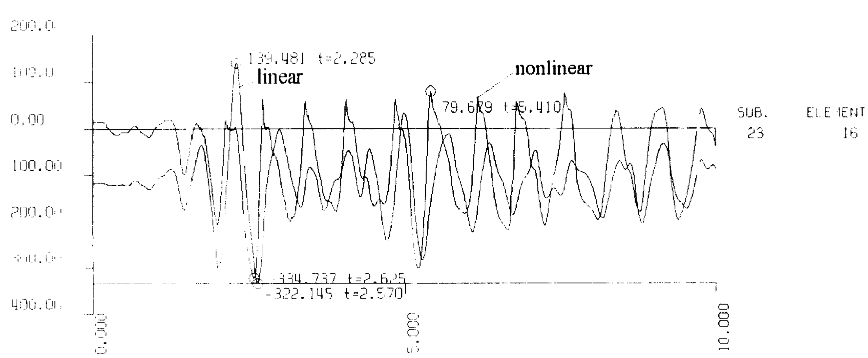


Fig. 8. Time history of componental stresses Gxy ts=0. te=10.sec.

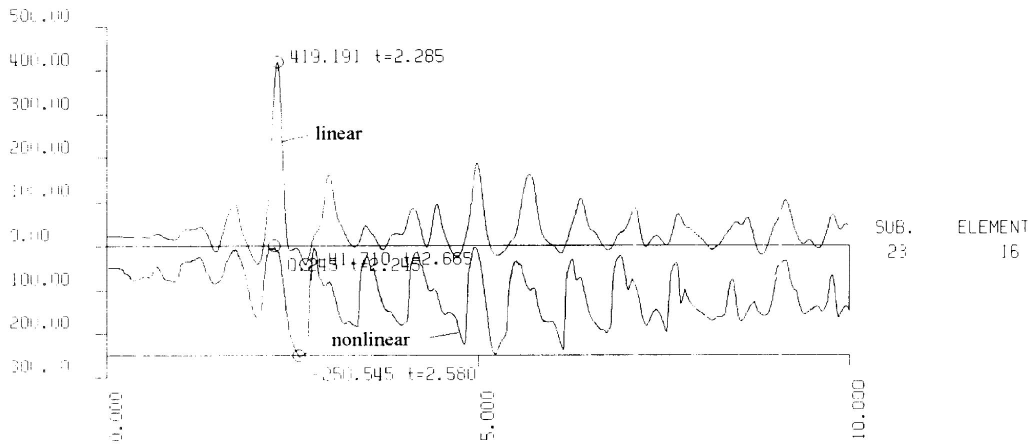


Fig. 9. Time history of principal stresses G1 ts=0. te=10.sec.

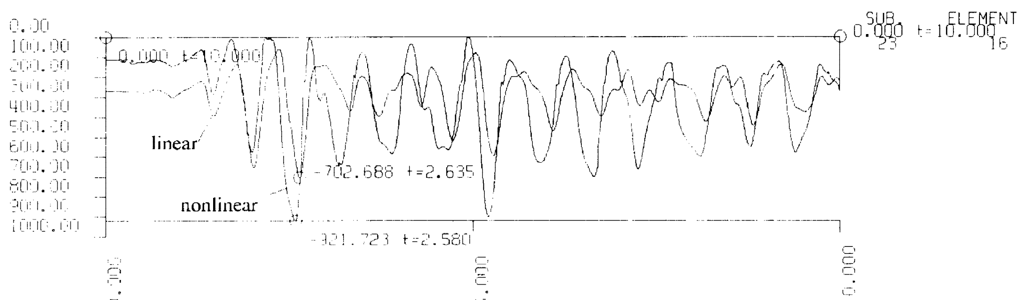


Fig. 10. Time history of principal stresses G2 ts=0. te=10.sec.

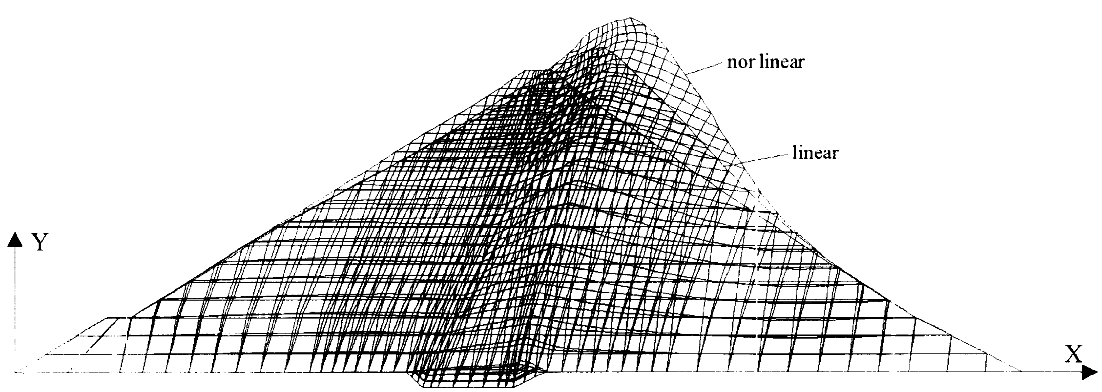


Fig. 11. Snapshot T=2.26 sec. Xmax=0.191 m
Imperial Valley Ax=0.36g Ay=0.27g
Max. linear deformation X=0.12m Y=0.047m
Max. nonlinear deformation X=0.191m Y=0.105m

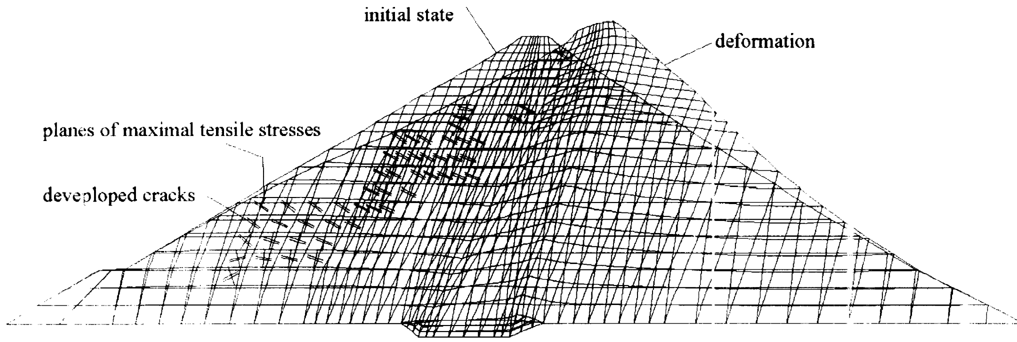


Fig. 12. Location of the developed cracks T=2.205 sec
Imperial Valley Ax=0.36g Ay=0.27g

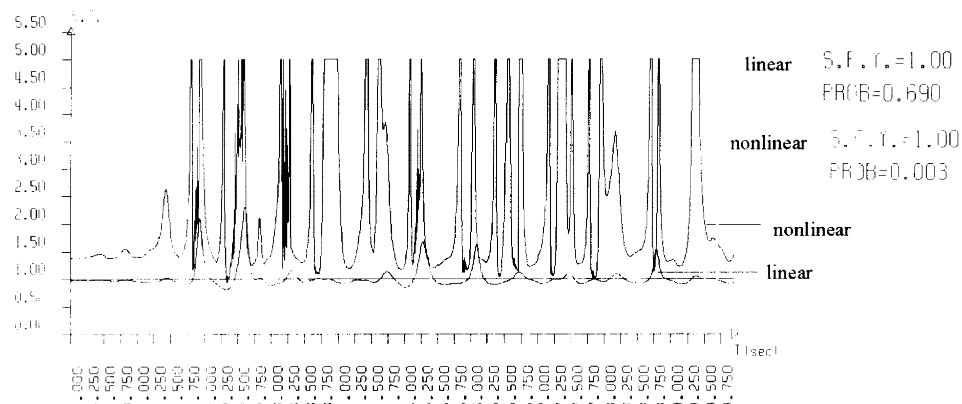
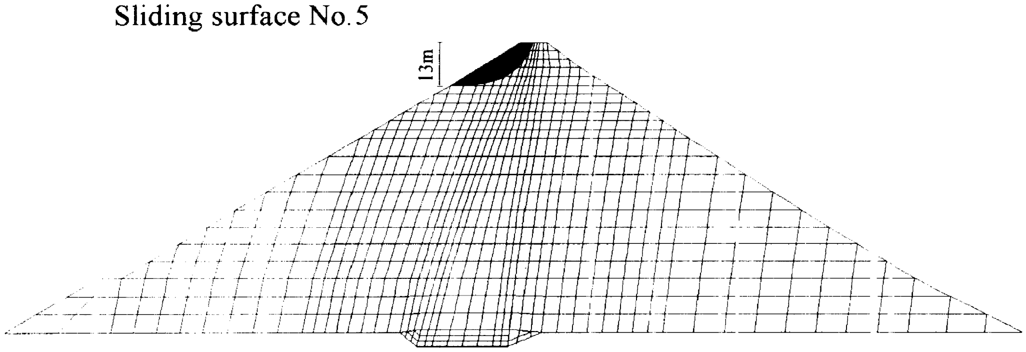


Fig. 13. Time history of the safety coefficient
Sliding surface No.5
Imperial Valley Ax=0.36g Ay=0.27g

Sliding surface No.5



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