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ANALYSIS OF LIQUEFACTION INDUCED DISPLACEMENTS IN EARTHDAMS

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ABSTRACT

The seismic security assesment of earthdams requires not only the final stability assesment but also the evaluation of the seismic caused deformation and cracking.
A method is proposed in this paper to evaluate the deformation needed to reach post-liquefaction equilibrium under gravity loads. As an example some results are shown for Cuesta del Viento Dam which is located at Jachal River in San Juan Province, Argentina
The post-licuefaction deformations needed to reach a new state of static equilibrium under gravity loads are computed, in a total stress aproach, using Duncan's model with hyperbolic stress-strain laws for the dam materials. The stress state before the earthquake is used as the initial stress state to compute the parameters of the hyperbolic stress-strain law for each element of the finite element mesh. The parameters of the licuefied zone stress-strain curves are taken from consolidated undrained tests.

KEYWORDS

Earthdams; licuefaction; displacements; post-licuefaction; freeboard; deformations

INTRODUCTION

The earthquake security of earthdams is highly influenced by the strength loss due to liquefaction when the submerged upstream shell or foundation materials are sensitive to this phenomenon. After the recognition of this fact in the 60's, efforts were directed to develop analytical tools for the evaluation of the liquefaction hazard and the prevention of large pore pressure built-up was adopted as a design crtiteria (Seed, 1966, 1969, 1981). This design criteria seems to be very safe, since the prevention of pore pressure built-up in the dam ensures that no deleterious effects associated to liquefaction will occur (Seed, 1986).

The security analysis of earthdams against liquefaction when very strong accelerograms are used as design motion (e.g. Fig. 5), such as could be expected in epicentral regions with high recurrence periods, showed that almost every material, even well compacted gravels, would liquefy during such an earthquake. Since at present, no effective remedial measures for liquefaction are known, the non-liquefaction requirement is imposible to fulfil for such cases.

On the contrary, in the last decade most researchers recognized that gravely soils compacted at high relative densities, have high residual (steady state) strength after liquefaction, since shear straining causes dilatation and the inmediate drop of the pore pressures (Castro, 1989; De Alba, 1987; Ishihara, 1990; Markuson, 1990). This fact determines that the final stability of dams constructed with such materials is ensured for most earthquakes, even if liquefaction takes place. However, the seismic security assesment of the structure requires not only the final stability assesment but also the evaluation of the seismic caused deformation and cracking. Deformations can cause loss of freeboard with overtopping hazard, and cracking can lead to tubification. Both situations can affect the overall structure security.

Deformation of earthdams due to seismic actions are caused by different and complex phenomena. Some deformation types caused by earthquakes are:

- Shear deformations due to inertia forces.
- Shear deformations under gravity loads due to loss of strength and stiffness caused by liquefaction.
- Volumetric deformation due to post-liquefaction consolidation with drainage.

These types of deformations are coupled within the earthquake duration and also after it has stopped. The displacements caused by the first type of deformation can be estimated with a Newmark type analysis. A method is proposed in this paper to evaluate the second type of deformations, which are needed to reach post-liquefaction equilibrium under gravity loads. The method does not intend to describe accurately the complex behavior of the dam but rather to serve as a design tool and allow to estimate the displacements which could be caused by the earthquake. The method is applicable only if the structure is stable considering the materials residual strength.
As an aplication example, some results are shown for Cuesta del Viento Dam which is located at Jachal River in San Juan Province, Argentina.

ESTIMATION OF DISPLACEMENTS DUE TO LIQUEFACTION

The behavior of an earthdam with submerged zones under severe earthquake action comprises several phenomena which are coupled in a complex manner. The proposed method is based on the following interpretation of this behavior.

The earthquake action causes pore pressure build-up in certain zones of the dam where materials susceptible to liquefaction are saturated. The consecuence of pore pressure built-up is the loss of strength and stiffness, but significant permanent deformations of the liquefied zones do not take place inmediately since they are limited by the non-liquefied stiffer rest of the structure. Only if the growing of this zone is such that the static equilibrium under vertical and inertial loads is not posible, a slide with large plastic deformations can take place. During the liquefaction process, the deviatoric stress decreases in the liquefied zone due to loss of stiffness.

Once the unstable condition is reached the structure deforms involving a stress redistribution. Deformations continue until the materials reach the strength value, a fraction of the residual strength, needed to restore the equilibrium namely a new equilibrium configuration under gravity loads. During the deformation process, saturated materials should follow stress-strain relations similar to those obtained from consolidated-undrained triaxial tests, and non-saturated materials to those obtained from drained triaxial tests.

The stress-strain states during the deformation process is showed schematically in Fig. 1 for the liquefied material. Point (1) corresponds to the state before liquefaction, and it lays on a "drained" stress-strain curve. Point (3) corresponds to the final state, when the deformation process has stopped and the redistribution of stresses has occured, and it lays on an "undrained" stress-strain curve.

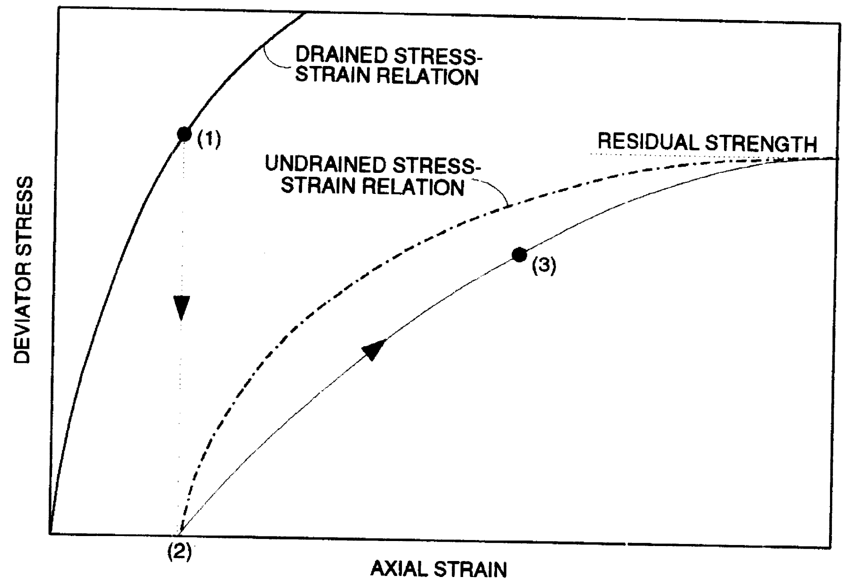

Fig. 1 Stress-strain states during the deformation process.

The actual path between points (1) and (3) is unkwown. The proposed method assumes that the deformation path is (1)-(2)-(3). Figure 2 shows actual stress-strain curves obtained from drained and consolidated undrained triaxial tests for the *Cuesta del Viento* gravel. The material's residual strength is nearly so high as the drained strength (greater than 80%), but the initial stiffness of both curves are very different. In addition, initial stiffness from undrained triaxial tests carried out on previously liquefied probes are lower than the stiffness of non-licuefied ones. This behavior was reported by Banerjee *et al* (1979) and it is shown in Fig. 3. These observations mean that significant displacements could be expected if the material liquefies.

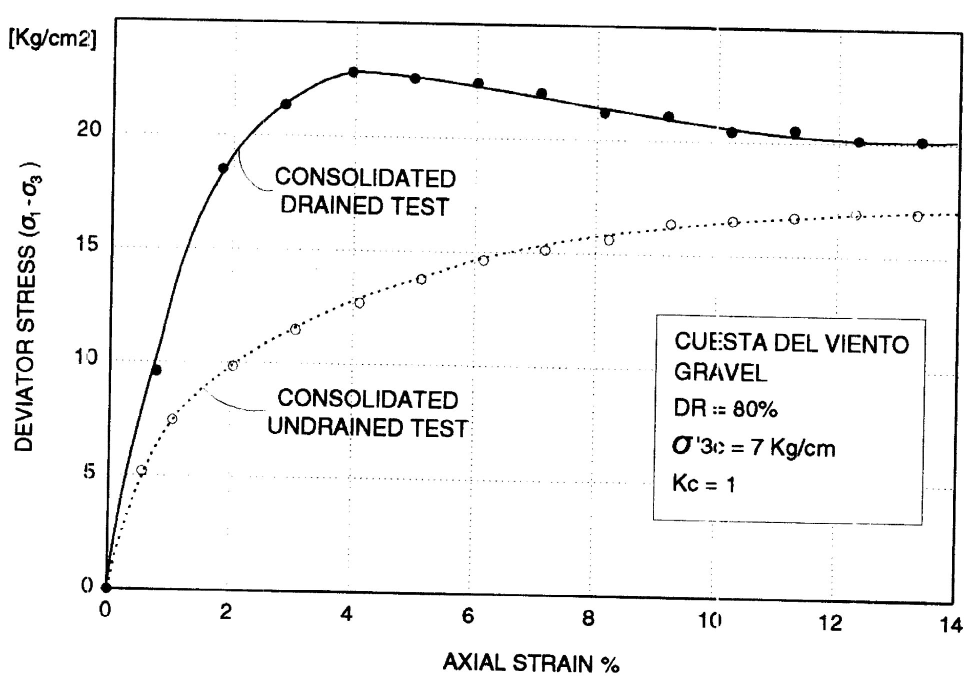

Fig. 2. Triaxial tests of the Cuesta del Viento Gravel.

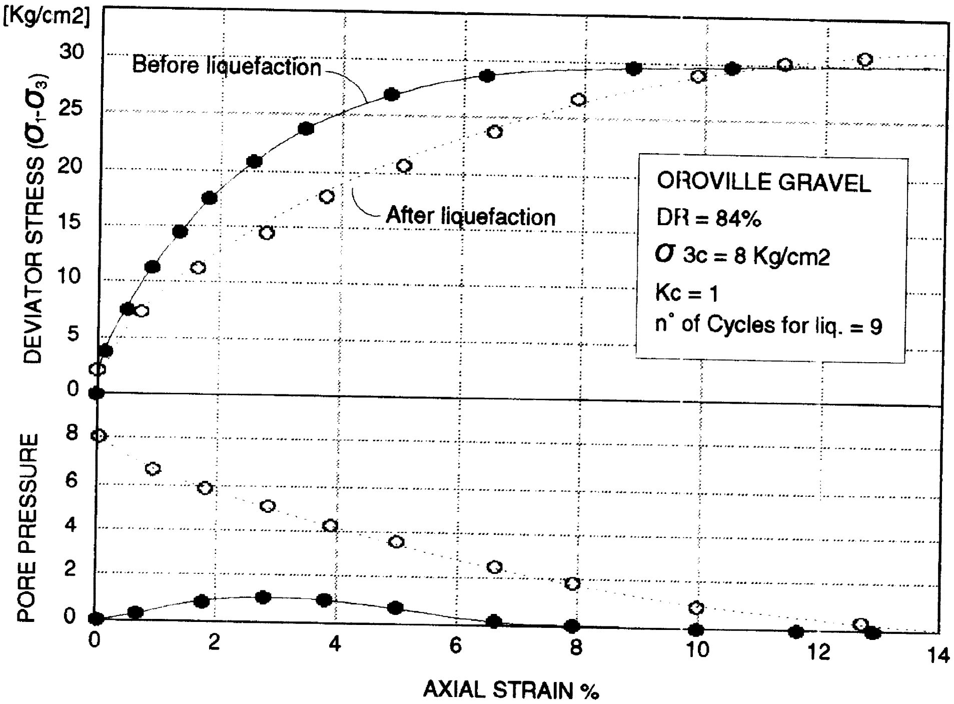

Fig. 3. Triaxial tests of the Oroville Gravel. (Banerjee *et al*, 1979)

The proposed method comprises three main analysis steps:

Step 1: The stress distribution under gravity and reservoir hydrostatic loads is computed by means of a finite element model using hyperbolic constitutive laws proposed by Duncan (1980).

Step 2: The pore pressure distribution caused by the design earthquake is evaluated. The zones of the dam where high residual pore pressures have developed are assumed to be liquefied (conventionaly where ru >80%). It is assumed that no displacement take place in the structure until the liquefied zones are large enought to make the slope unstable and also that the deviatoric stress drops to zero in the liquefied zone.

Step 3: The analysis of the dam displacements is done by means of a finite element model considering gravity and reservoir hydrostatic loads and hyperbolic stress-strain relations for the materials, in a total stress a aproach. The deformation process of saturated materials is asssumed to take place at constant volume (undrained) and to follow stress-strain relations similar to those obtained from consolidated-undrained triaxial tests. The non-saturated materials are assumed to have a stress-strain relation similar to those obtained from triaxial drained tests.

Displacements are computed by solving the equilibrium equation:

$$[\mathbf{K}].\{\mathbf{u}\}=\{\mathbf{f_B}\}+\{\mathbf{f_T}\}-\{\mathbf{f_I}\} \quad (1)$$

where :

$\{\mathbf{u}\}$ = nodal displacements vector

$[\mathbf{K}]$ = structure non-linear stiffness matrix = $\Sigma.\ [\mathbf{K^e}]$

The element stiffness matrix $[\mathbf{K^e}]$ is obtained using the pre-earthquake stress distribution (computed in Step 1) as initial state for computing the parameters of the stress-strain law for each element of the finite element

mesh. The considered constitutive laws are:
- For non-saturated elements, hyperbolic stress-strain laws fitted to drained triaxial tests results.
- For saturated and non-liquefied elements, hyperbolic stress-strain laws fitted to consolidated-undrained triaxial tests results. The pre-earthquake static stress state is considered as the consolidation state.
- For saturated and liquefied elements, same as non-liquefied elements but using a reduced initial tangent elasticity modulus (a fraction of the one observed in the tests). As an alternative, the initial modulus can be measured from static undrained tests after liquefaction of the specimen.
Saturated materials are assumed to be incompressible.

$\{\mathbf{f_B}\}$ = nodal forces due to body forces, i.e. the material weigth = $\Sigma.\ \{\mathbf{f_B}^e\}$
The element body forces $\{\mathbf{f_B}^e\}$ are computed for each element as follows:
- For non-saturated elements, using the material unit weight.
- For saturated elements, using the material submerged unit weigth.

$\{\mathbf{f_T}\}$ = nodal forces due to the hydrostatics pressures from reservoir water = $\Sigma.\ \{\mathbf{f_T}^e\}$

$\{\mathbf{f_I}\}$ = nodal forces due to initial stresses, i.e. the pre-earthquake stress state= $\Sigma.\ \{\mathbf{f_I}^e\}$
The element forces $\{\mathbf{f_I}^e\}$ are computed for each element as follows:
- For non-liquefied elements, using the pre-earthquake stress distribution.
- For liquefied elements, using the hydrostatic part of the pre-earthquake stress distribution (assuming zero deviatoric stress).

The load vector rows becomes equal cero for nodes within the non liquefied zone since the initial stress state equilibrates the gravity and hydrostatic loads. Since the used constitutive laws are non-linear, eq. (1) is solved incrementally, splitting the second member of eq. (1) in small load steps.

SEISMIC SECURITY EVALUATION

The seismic security evaluation of Cuesta del Viento Dam was carried out using the following methodology:
a.- Selection of the verification accelerogram. This was derived from a 7.3-7.7 Mw magnitude earthquake at a short epicentral distance.
b.- Estimation of the liquefied zone using dynamic analysis, materials cyclic strength values and Standard Penetration Test correlations.
c.- Limit equilibrium check under gravity load using material residual strength.
d.- Post-liquefaction displacement estimation using static nonlinear analysis.
e.- Estimation of displacement due to inertia forces using Newmark's sliding block analysis.

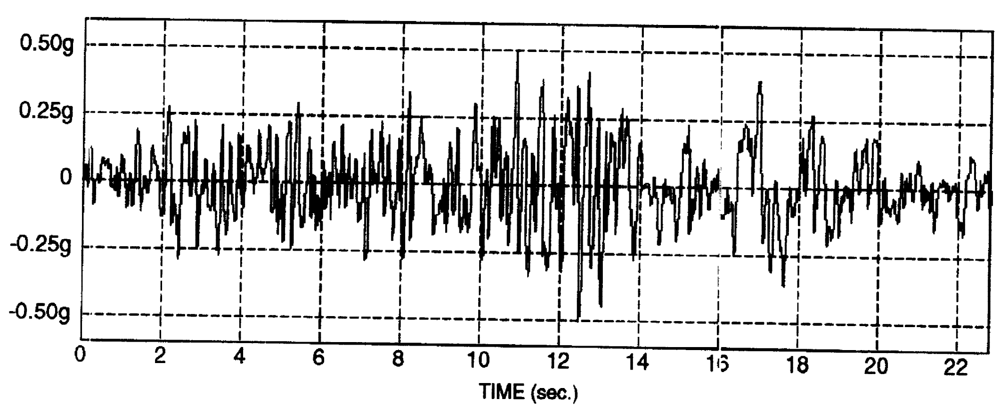

Fig. 4 Verification accelerogram

The postulated accelerogram is shown in Fig. 4. This motion has a maximum acceleration of 0.52 g and would cause the liquefaction of almost all the upstream shell and its foundation material. Figure 5a shows a section of the dam.

Figure 5b shows the computed displacement pattern for Cuesta del Viento Dam using the proposed approach. The computed loss of freeboard was 1.3 m. The displacement increment at the last step of the analisys can be seen in Fig. 5c which illustrates the nature of the slip motion.

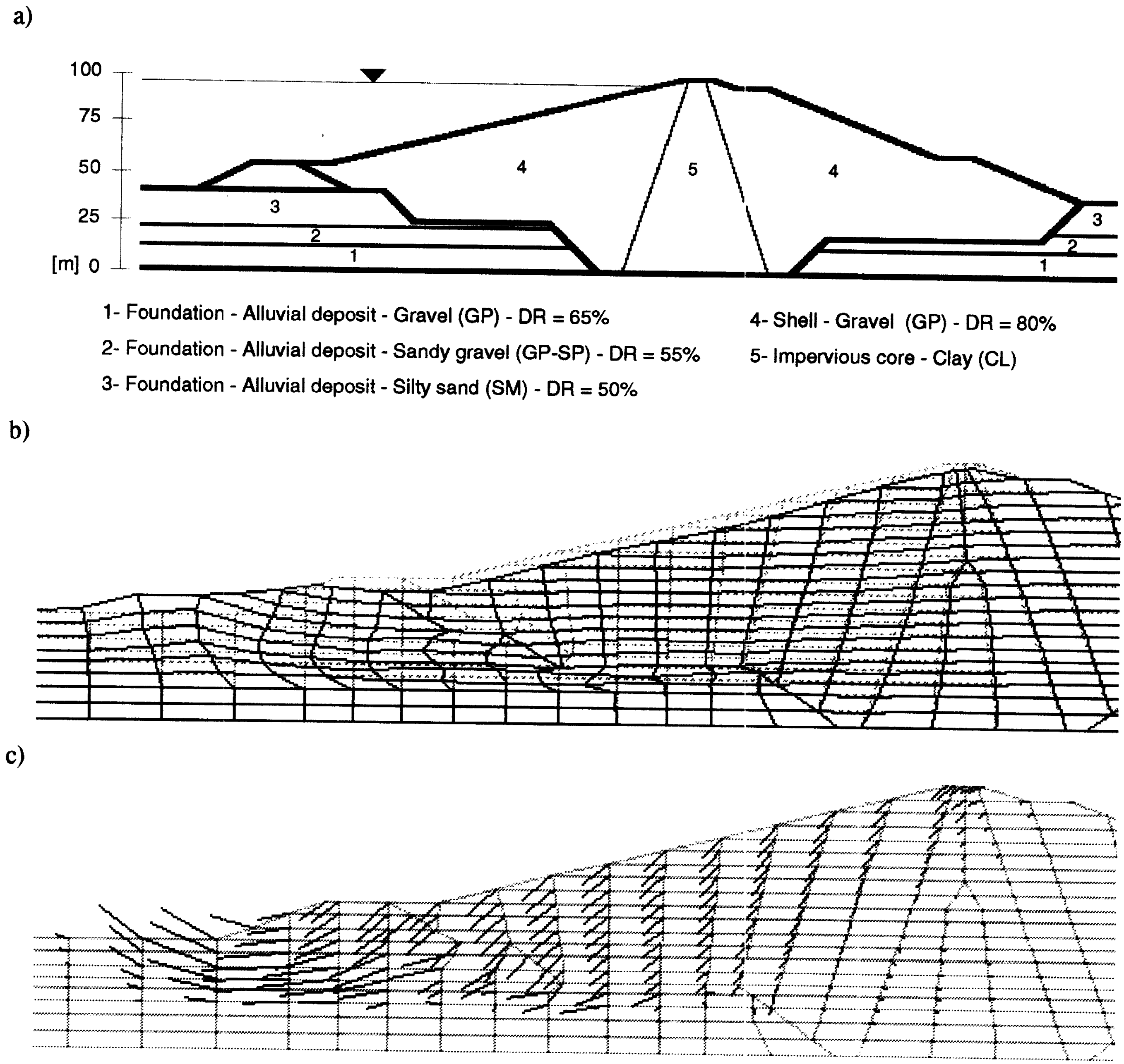

Fig. 5. a) Cuesta del Viento Dam analysis section
b) Computed displacement pattern (displacements scale 3:1).
c) Displacement increment at the last load step.

The displacement value due to inertial forces, estimated with a Newmark type analysis was 0.55 metres. Since the dam was designed with 8 metres of freeboard above the normal operation reservoir level, it was concluded that the occurrence of an earthquake similar to the aplied design motion can not cause overtopping hazard.

CONCLUSIONS

The seismic security evaluation of earthdams requires not only the stability assesment of the slopes but also the estimation of permanent deformations caused by seismic action.

The methodology presented in this paper can be used to estimate post-liquefaction displacements and to evaluate the security of design decisions and remedial measures. Also it can be used to make sensitivity analysis varying materials residual strengths and stiffness. For extremely strong earthquakes it is possible for the unstable condition to take place more than once during the motion. For this cases, the actual loss of freeboard may be greater than the one computed with the proposed method and judgement is required for the results interpretation.

Considering the scarcity of actual behavior records of earthdams during earthquake action, it is very difficult to formulate and verify accurate mathematical models for security analysis. For that reason, the proposed method does not intend to describe accurately the complex behavior of the dam but rather to serve as a design tool allowing to estimate how the earthquake could affect the dam security.

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