Paper No. 1046. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

EVALUATION OF THE LIQUEFACTION POTENTIAL AT TUMACO (COLOMBIA) USING AN EFFECTIVE STRESS APPROACH

A. LEDESMA[1] and J. GARCÍA[2]

[1] School of Civil Engng., Technical University of Catalunya, UPC
08034 Barcelona, SPAIN
[2] INGEOMINAS, Diagonal 53 n. 34-53, Bogotá, COLOMBIA

ABSTRACT

The paper presents the application of two procedures to evaluate the liquefaction potential at Tumaco (Colombia). The simplified procedure proposed by Seed was used in 1993 by INGEOMINAS to estimate such risk. Now an effective stress approach based on the numerical formulation by Zienkiewicz has also been used, and the results of its application are shown. Results obtained by both procedures are consistent, but the effective stress approach gives information on the evolution of pore water pressures, displacements and accelerations, including the consolidation phase after the earthquake.

KEYWORDS

Liquefaction potential, effective stress approach, finite element method, coupled analysis.

INTRODUCTION

Tumaco is located on the Pacific coast of Colombia, close to the border with Ecuador (fig. 1a). This area is well known for its seismic activity due to the near contact between the Nazca plate and the Southamerican plate. According to the Colombian seismic code (Código Colombiano, 1984), the zone corresponds to a high seismic risk area, with an expected maximum base rock acceleration of 0.3g. Due to the scattering of the data available, conservative values have been usually adopted for design purposes. Recently, the maximum expected earthquake magnitude in the area has been estimated as $M_s = 8.5$, and the corresponding surface ground acceleration close to 0.45g, allowing for a certain amplification factor from the basement acceleration (INGEOMINAS, 1993). From a geological point of view, the area of Tumaco is located on the delta of Mira river and therefore the stratigraphy corresponds mainly to sediments of saturated sands, silty sands and peats over the sandstone basement. As in some cases sands are in a loose state, the risk of liquefaction must be considered in any construction development.

A site investigation was carried out in 1993 on a particular area which was selected for a new housing development. That included boreholes with basic identification tests, standard penetration tests and geophysical exploration. The evaluation of the liquefaction potential was then performed by means of the simplified procedure proposed by Seed and Idriss (1971) (INGEOMINAS, 1993). As a consequence of that analysis, the report suggested to use part of the land for recreational areas, due to the high risk of liquefaction encountered, and pile foundations were proposed for new buildings in some zones. In despite of the definite results of this analysis, the area of Tumaco has been selected to develop a reserch programme to improve the determination of the liquefaction potential in sandy deposits, which are usually found in the Pacific Colombian coast. As a first step, an effective stress approach based on the formulation by Zienkiewicz *et al* (1990a,b) has been alternatively used. That formulation has proved to be a good framework to analyze such problems. Using this approach, a finite element code for solving one dimensional soil columns under cyclic loading has been developed, similar to that used by Chan (1988) and Xie (1990). The program has been used to analyze some of the soil profiles from Tumaco considered in the simplified procedure. Also, cyclic triaxial and resonant column tests were carried out in order to complete the definition of the sand properties. In this way, two independent methodologies will have been used to assess the risk of liquefaction of the area.

First a brief report of the results obtained using the simplified approach is presented. Next the effective stress approach is outlined, including the basic equations and the numerical solution employed. Finally, the application of the formulation to the soil profile of Tumaco is presented, and the results are compared with those obtained by means of the simplified procedure.

SIMPLIFIED APPROACH

The method considered here as "simplified approach" corresponds to the one proposed by Seed and Idriss in 1971 which has been extensively used and improved for evaluating the liquefaction potential of numerous sites in the world. It is a straightforward procedure, but some of the assumptions used are based on empirical expressions. The method requires to evaluate the shear stress produced by the earthquake and to compare it with the shear stress level that produces liquefaction obtained from empirical relations with Standard Penetration Tests or from cyclic triaxial tests. A simple estimation of the seismic average shear stress referred to the *in situ* overburden effective stress, σ'_o, is:

$$\frac{\tau_{av}}{\sigma'_o} \approx 0.65 \, \frac{a_{max}}{g} \, \frac{\sigma_o}{\sigma'_o} \, r_d \tag{1}$$

where a_{max} refers to the maximum acceleration considered, g is the gravity acceleration, σ_o is the total overburden stress and r_d is a coefficient defined in Seed and Idriss paper. On the other hand, the estimation of shear stress that produces liquefaction in a particular soil, τ_{lic}, from "N" values (corrected from the measured values in the SPT test), can be obtained from the empirical relations given by Seed (1979). This shear stress is also referenced to the *in situ* overburden effective stress.

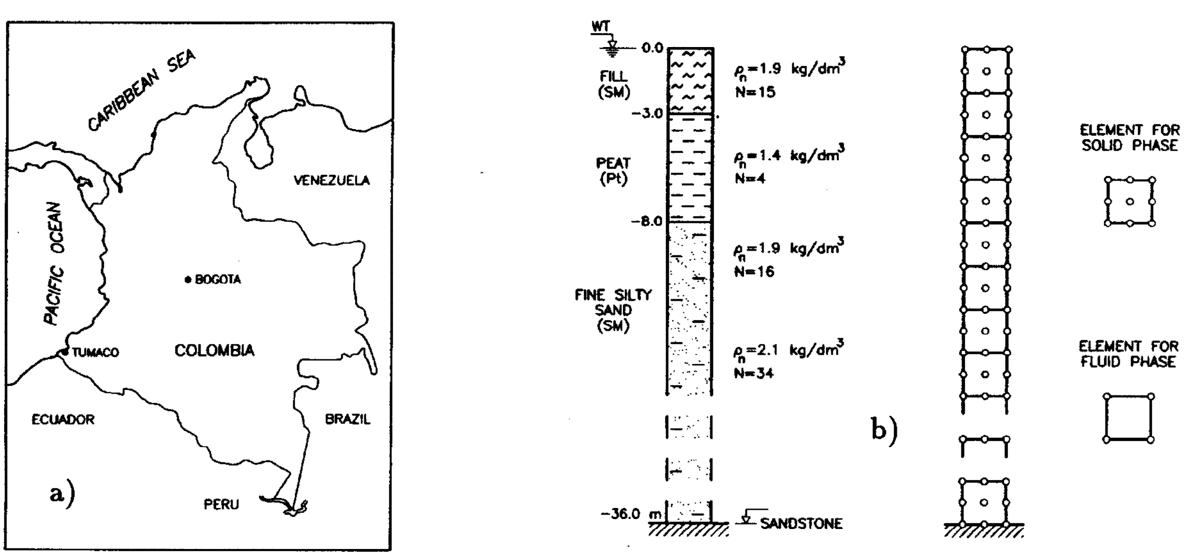

Fig. 1. a) Location of Tumaco. b) Soil profile and finite element mesh.

Finally, comparing both stress ratios, a safety factor for liquefaction can be estimated:

$$FS = \frac{\tau_{lic}/\sigma'_o}{\tau_{av}/\sigma'_o} \tag{2}$$

In this case, the soil analyzed is represented in fig. 1b, where a typical profile is shown, including the SPT blow count and the density. The value of $1900 Kg/m^3$ corresponds to a relative density close to 30% for this sand. The top layer is an artificial fill made by silty sand from the area. Its grain size is quite uniform, with a uniformity coefficient of 2.5 and 95% finer by weight than the ASTM 40 sieve (0.425 mm). By means of equations (1) and (2), and considering the corrections suggested by Seed (1979), the following results were obtained (borehole nr. 7, from INGEOMINAS report, 1993):

- *Case 1:* Earthquake with magnitude $M_s = 8.5$, $a_{max} = 0.43g$
  Depths considered: 1.5 m , 9.5 m , 12.5 m
  FS obtained: 0.5, 0.3, 0.6, respectively
- *Case 2:* Earthquake with magnitude $M_s = 7.5$, $a_{max} = 0.4g$
  Depths considered: 1.5 m , 9.5 m , 12.5 m
  FS obtained: 0.6, 0.4, 0.7, respectively.

Therefore, the sandy layers close to the surface (at least 12 to 15 m) were suitable of liquefaction under these earthquakes, considered probable in the life of the houses to be built in the area. Note that safety factors are not close to the "safe value = 1 ", that is, liquefaction with weaker earthquakes could be expected in this soil column. Due to this result, some restrictive conditions were suggested for the housing scheme design. However, as this situation is quite common in the area, additional liquefaction analysis would be convenient to assure these results and in this way, an effective stress approach has been considered a convenient method to improve the quality of the liquefaction potential analysis.

EFFECTIVE STRESS APPROACH

Basic equations

The basic equations describing the wave propagation in an elastic porous media were initially presented by Biot in a general form (1956). A general formulation of the equations in a form suitable for a numerical solution using the Finite Element Method was presented by Zienkiewicz and Shiomi (1984). Further upgrades of the formulation, including in a simple way the effect of the unsaturated soil, have been described in Zienkiewicz *et al* (1990a,b). Assuming small deformations and using the basic principles of Continuum Mechanics, the mathematical relations involved in this problem are the following:

Effective stress principle. The effective stress is defined as:

$$\sigma'_{ij} = \sigma_{ij} + p\delta_{ij} \tag{3}$$

where σ_{ij} is the Cauchy stress tensor, δ_{ij} is the identity matrix and p is the pore water pressure. In (3) tensile stresses are assumed positive, whereas water pressure is positive in compression.

Small strains tensor. If u_i is the ith component of the soil phase displacement:

$$\epsilon_{ij} = \frac{1}{2}\left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i}\right) \tag{4}$$

Constitutive law. A nonlinear relation between increments of effective stresses and strains is defined:

$$d\sigma'_{ij} = D_{ijkl} d\epsilon_{kl} \tag{5}$$

where D_{ijkl} will depend on the stress state and some history parameters.

Momentum equilibrium equation. Assuming that coordinate system moves with the solid phase:

$$\frac{\partial \sigma_{ij}}{\partial x_j} + \rho b_i = \rho \ddot{u}_i + \rho_f \ddot{w}_i \tag{6}$$

where b_i represents the ith component of the vector of body force per unit volume, and $\ddot{u}_i$, $\ddot{w}_i$ represent the components of the accelerations of the solid and fluid phase respectively. ρ is the soil density and ρ_f the fluid one.

Momentum equilibrium for fluid phase. Including the Darcy's law which is assumed valid under dynamic conditions:

$$\dot{w}_i = -k_{ij}\frac{1}{\gamma_f}\left[\frac{\partial p}{\partial x_j} - \rho_f(b_j - \ddot{U}_j)\right] \tag{7}$$

where $\dot{w}_i$ refers to the velocity of water, k_{ij} is the permeability tensor, $\gamma_f = \rho_f g$, and

$$\ddot{U}_j = \ddot{u}_j + \frac{D}{Dt}\left(\frac{\dot{w}_j}{n}\right) \tag{8}$$

In this expression, the sign D/Dt means material derivative, and n is the soil porosity.

Water continuity equation. It can be written as:

$$-\frac{\partial \dot{w}_i}{\partial x_i} = \frac{\partial \epsilon_{ii}}{\partial t} + \frac{1}{Q}\frac{\partial p}{\partial t} \tag{9}$$

where $1/Q = n/\mathcal{K}_\mathrm{f} + (1-n)/\mathcal{K}_\mathrm{s}$, and $\mathcal{K}_\mathrm{f}, \mathcal{K}_\mathrm{s}$ are the bulk moduli of the fluid and soil grains respectively. Usually $\mathcal{K}_\mathrm{s} >> \mathcal{K}_\mathrm{f}$ and its effect is neglected.

Formulation based on displacements and water pressure.

In previous equations, some approximations have been assumed. For instance fluid convective terms have been neglected and densities have been considered constant, although in (9) the compressibility of water and solid particles is somehow considered. An additional reasonable assumption in earthquake analysis is to consider relative fluid accelerations small if compared with solid accelerations. In that case $\ddot{w}_i$ can be neglected and $\dot{w}_i$ could be removed combining equations (7) and (9). Hence, the final set of equations is:

$$\begin{aligned}
d\epsilon_{i,j} &= \frac{1}{2}(du_{i,j} + du_{j,i}) \\
\sigma'_{ij} &= \sigma_{ij} + p\delta_{ij} \\
d\sigma'_{ij} &= D_{ijkl}d\epsilon_{kl} \\
\sigma_{ij,j} + \rho b_i &= \rho\ddot{u}_i \\
\ddot{\epsilon}_{ii} + \frac{n}{\mathcal{K}_\mathrm{f}}\dot{p} &= \left(\frac{k_{ij}}{\gamma_f}\frac{\partial p}{\partial x_j}\right)_{,i} - \left(\frac{k_{ij}}{g}b_j\right)_{,i} + \left(\frac{k_{ij}}{g}\ddot{u}_j\right)_{,i}
\end{aligned} \tag{10}$$

which is the so called **u - p** formulation (Zienkiewicz *et al* , 1990a), because the main variables are the solid displacements and the pore water pressure. These equations represent the pore solid - fluid interaction and its application can be extended to nondynamic conditions i.e. during consolidation after the earthquake. It should be pointed out that equations (10) are coupled and that a soil constitutive law must be introduced in the third expression of this set.

In this paper, the Pastor - Zienkiewicz model has been used as constitutive law. It is an elastoplastic model based on the generalised plasticity approach and it does not require the explicit definition of a yield surface or a plastic potential, but only a definition of the normal vectors to those surfaces in any point of the stress space. Other features of the model include a hierarchical structure and hardening parameters and unloading response depending on both volumetric and deviatoric plastic strains. A detailed description of the model can be found in Pastor and Zienkiewicz (1986), where a good agreement between experimental and numerical results is also shown. Parameters of the model used in this analysis have been obtained from experimental tests. In particular, resonant column tests and cyclic triaxial tests were performed on Tumaco sand in order to estimate their values. Figures 2a and 2b show some of the experimental results.

Numerical solution.

The set of equations (10) using the model indicated can be solved using a suitable discretization technique. Following Zienkiewicz *et al* (1990a), the finite element method was used for the spatial discretization, obtaining:

$$\begin{aligned} \mathbf{M}\ddot{\bar{\mathbf{u}}} + \mathbf{K}\bar{\mathbf{u}} - \mathbf{Q}\bar{\mathbf{p}} &= \mathbf{f^u} \\ \mathbf{Q^t}\dot{\bar{\mathbf{u}}} + \mathbf{S}\dot{\bar{\mathbf{p}}} + \mathbf{H}\bar{\mathbf{p}} &= \mathbf{f^p} \end{aligned} \tag{11}$$

where $\mathbf{M}$ is the mass matrix, $\mathbf{Q}$ is the coupling matrix, $\mathbf{S}$ is the compressibility matrix, $\mathbf{H}$ is the permeability matrix and $\mathbf{K}$ is the stiffness matrix which includes the constitutive law and therefore depends on the stress state. Also, $\mathbf{f^u}$ represents a force vector and $\mathbf{f^p}$ a water flow vector, and they include boundary conditions; and $\bar{\mathbf{u}}$, $\bar{\mathbf{p}}$ represent the vectors of nodal displacements and nodal water pressures respectively. Description of these matrices and vectors can be found in Zienkiewicz *et al* (1990a). Note that system of equations (11) does not include a term on $\dot{\mathbf{u}}$, as the solid – fluid interaction provides a natural damping effect. However, an additional term could be incorporated if necessary. Time discretization in (11) was performed by means of a Newmark scheme (finite difference technique) and then (11) becomes a nonlinear system of algebraic equations for each time.

Application to Tumaco

Figure 1b includes the finite element mesh used for spatial discretization of the soil column analyzed. Note that 9 noded and 4 noded quadrilateral elements have been used for interpolating displacements and pore water pressure respectively. Repeated boundary conditions were imposed so displacements are the same on the left side and on the right side of the column. The base of the column is supposed fixed to the rock basement, where the earthquake input acceleration is applied.

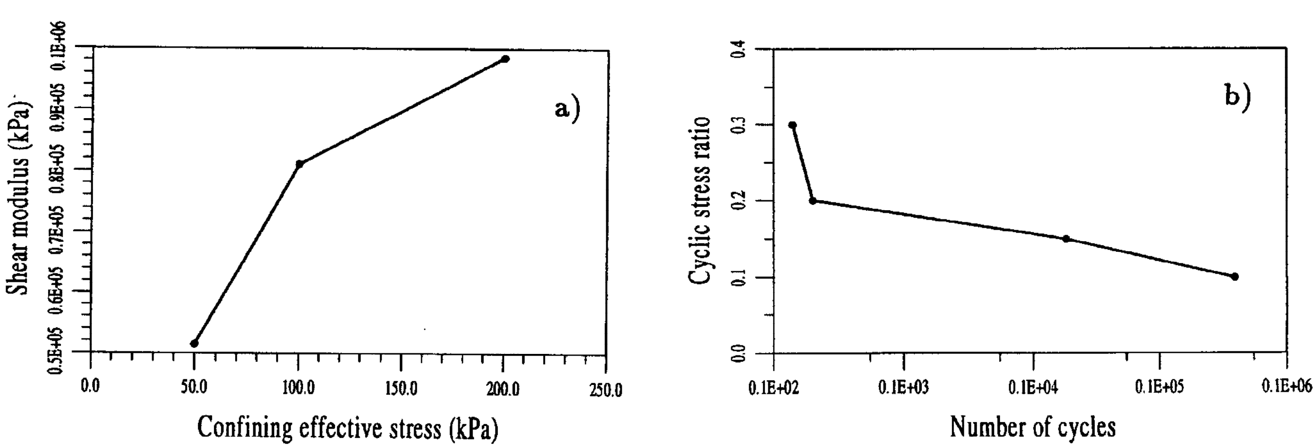

Fig. 2. a) Shear moduli from resonant column tests. b) Cyclic stress ratio causing liquefaction ($\sigma_{dev}/2\sigma'_{conf}$) for $\rho = 1950 Kg/m^3$ in cyclic triaxial tests.

In this analysis, an acceleration record obtained in a similar geological basement, but in another point of the Colombian Pacific area has been used. It corresponds to the Calima's earthquake, recorded by the INGEOMINAS station in Popayán, last August, 2nd, 1995, and had a magnitude Ms = 6.6. Figure 3 shows the east-west and the vertical components of the acceleration. North - south component has also been used in the analysis, but shows smaller values of acceleration. The duration was 17.31 seconds and the maximum acceleration is close to $6 cm/s^2$. This is a small value and no liquefaction could be expected due to this earthquake. However, as it is one of the best records obtained in the area, we have used it to perform a basic analysis. Afterwards an amplification factor was used to increase accelerations until liquefaction was simulated by the code.

Figures 4a and 4b show the evolution of the excess pore water pressure in a point 12 m depth, obtained running the code based on the formulation outlined above. Fig. 4a shows the variation during the first 40 seconds where pore pressure increases and fig. 4b depicts the whole evolution, including the consolidating phase. For this particular point, the initial effective stress is 83 kPa, so no liquefaction is produced as the maximum value reached during the earthquake is close to 11 kPa.

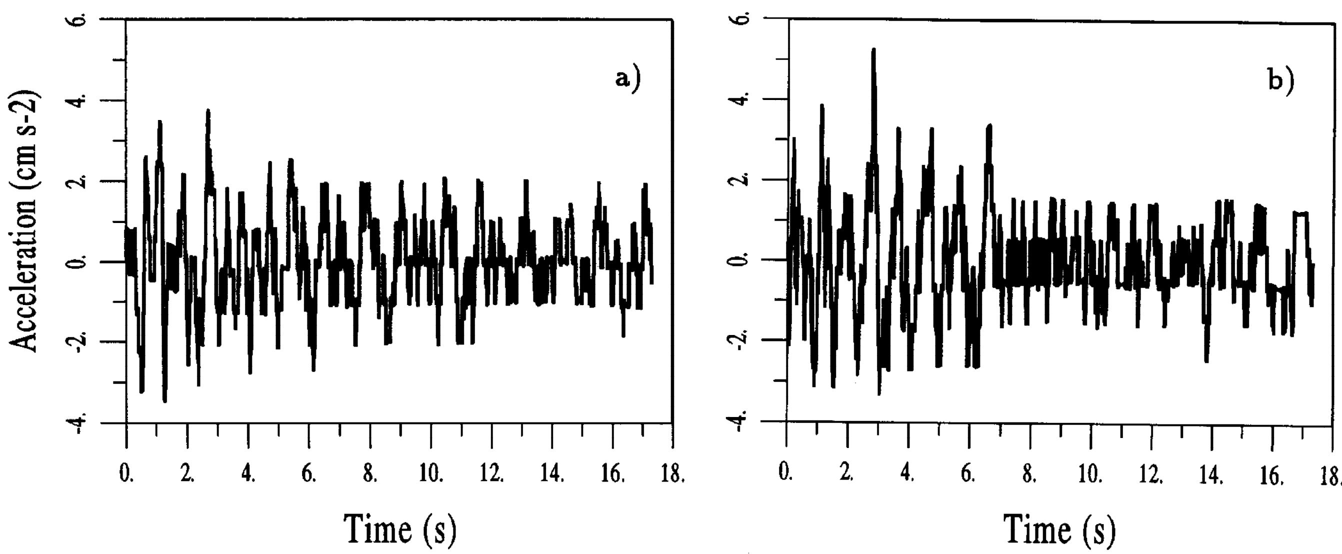

Fig. 3. Records of Calima earthquake: a) E-W component, b) vertical component.

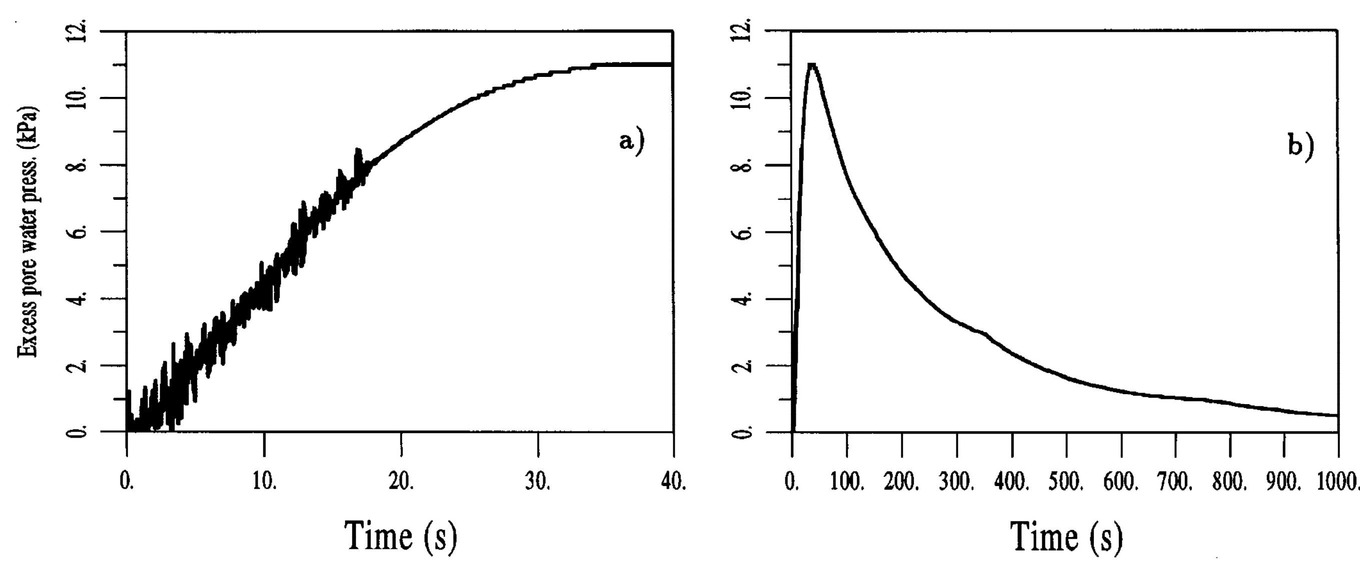

Fig. 4. Evolution of pore water pressure: a) generation, b) complete record.

The acceleration records shown in figure 3 were amplified until a significant liquefaction was reproduced. A factor of 6 was found to be enough to reproduce an increase of pore water pressure of 83 kPa for the same point. It must be pointed out that at 12 m depth the sand is still loose (natural density $\approx 1900 Kg/m^3$, relative density close to 30%), and deeper points increase their density to $2100 Kg/m^3$. That can explain the fact that even an amplification factor of 6 producing relatively small accelerations, can produce such increase in water pressure in the loose layers close to the surface. However, further investigations, including *in situ* and experimental tests will be carried out in the near future, to assure the validity of the quantitative results presented in this paper. Figure 5a presents the evolution of the excess of water pressure during the earthquake, compared with the overburden effective stress, for this case. Note that in 15 seconds the loose sandy layers close to the surface reach liquefaction. Figure 5b shows the settlement of the ground surface due to the earthquake. It can be seen that an important amount of displacement is produced after the earthquake, due to the consolidation process when pore pressure dissipates. In this case, 17 cm have been predicted.

Hence the effective stress approach predicts liquefaction for the soil of Tumaco as the simplified procedure does. However, figures 4 and 5 suggest that even for relatively small earthquakes, generation of pore water pressure and subsequent settlements could be produced. Note that in the simplified approach, accelerations of $0.4g$ were used in the analysis, whereas here the maximum value for the amplificated earthquake is $0.04g$. Also, in the effective stress approach, not only horizontal, but also vertical accelerations have been considered. These differences constitute an important drawback for comparing both analysis, in a quantitative way. Obviously the effective stress procedure is more sophisticated and gives more information on the earthquake effects, but also requires more data to be introduced on the computation procedure, so an additional effort in measuring field conditions and soil parameters must be performed.

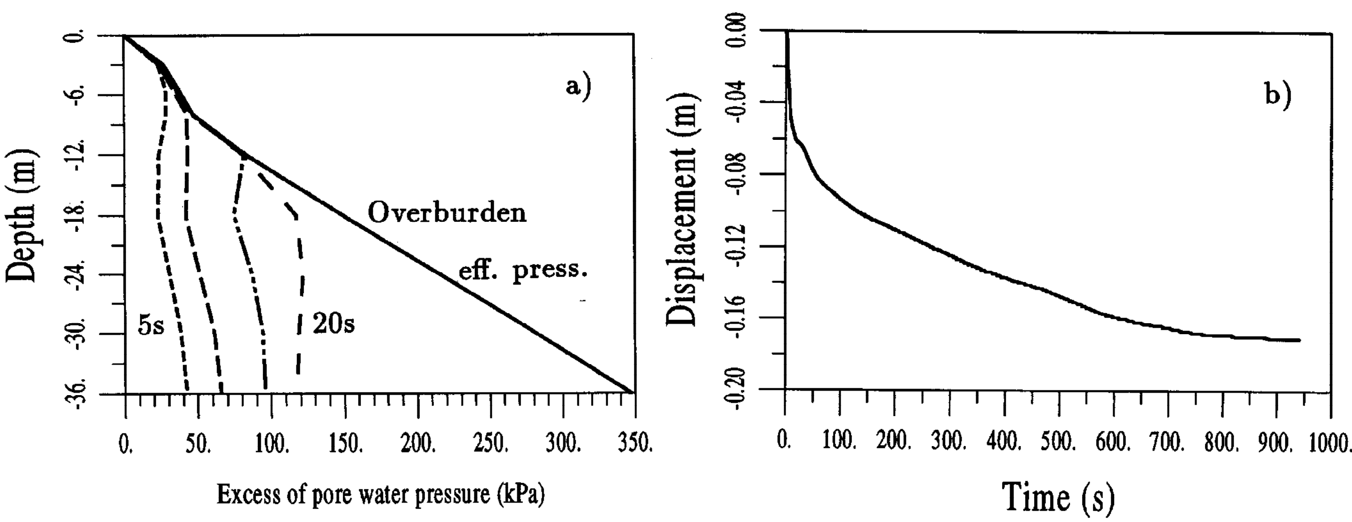

Fig. 5. a) Excess of pore water pressure profiles for different times: 5, 10, 15 and 20 s. b) Evolution of ground surface settlements.

CONCLUSIONS

Two methods for evaluating the liquefaction potential have been used in the area of Tumaco, in the Pacific coast of Colombia. Field tests and site investigations were performed in 1993 in order to estimate the liquefaction potential by means of a simplified procedure by Seed. This method allowed the identification of high risk areas. Afterwards, an effective stress approach based in a formulation by Zienkiewicz has been used. To do that, a 1D finite element code has been developed following that formulation and a typical soil column profile has been analyzed. Also, experimental tests carried out in the laboratory (i.e. resonnant column tests and cyclic triaxial tests) were used to define the soil properties.

Both methods of analysis have given consistent results, although, of course, the effective stress approach provides more information on displacements, evolution of water pressures and consolidation effects after the earthquake. According to both procedures, the soil profile analyzed has an important risk of liquefaction. In particular, the effective stress approach has reproduced liquefaction with what could be considered "small maximum accelerations". In despite of this results, additional work should be done in order to improve the calibration of model parameters and to ensure the validity of these analyses. In fact, this work is part of an ongoing research programme which includes the field measurement of accelerations and pore water pressures in Tumaco and its interpretation by means of an effective stress approach.

ACKNOWLEDGEMENTS

The authors would like to thank Mr. C. Mata, Mr. J. Suriol and Dr. A. Lloret, from UPC, for their collaboration in the experimental work.

REFERENCES

Biot, M. A. (1956) Theory of propagation of elastic waves in a fluid saturated porous solid. Part I: low frequency range. Part II: higher frequency range. *J. Acoust. Soc. Am.*, **28**, 168 – 191.

Chan, A. H. C. (1988). *A unified finite element solution to static and dynamic problems in geomechanics.* PhD thesis, University College of Swansea, U.K.

Código Colombiano de Construcciones Sismo-Resistentes (1984). *Decreto 1400.* Bogotá.

INGEOMINAS (1993). *Evaluación del potencial de licuación en Tumaco. Lote Inurbe.* Report. Bogotá, Colombia.

Pastor, M., and O. C. Zienkiewicz (1986). A generalised plasticity, hierarchical model for sand under monotonic and cyclic loading. In: *Numerical Methods in Geomechanics.* (Pande, G. N. and W. F. Van Impe eds), London, Jacson and sons. 131 – 150.

Seed, H. B. and I. M. Idriss (1971). Simplified procedure for evaluating soil liquefaction potential. *J. Soil Mechanics and Found. Div., ASCE*, **97**, 1249 – 1273.

Seed, H. B. (1979). Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes. *J. of the Geotech. Eng. Div. ASCE.* **105**, 201 – 255.

Xie, Y.M. (1990). *Finite element solution and adaptative analysis for static and dynamic problems of saturated - unsaturated porous media.* PhD thesis, University College of Swansea, U.K.

Zienkiewicz, O. C. and T. Shiomi (1984). Dinamic behaviour of saturated porous media: The generalised Biot formulation and its numerical solution. *Int. J. Num. Analyt. Meth. Geomech.*, **8**, 71 – 96.

Zienkiewicz, O.C., A. H. C. Chan, M. Pastor, D. K. Paul and T. Shiomi (1990a). Static and dynamic behaviour of soils: a rational approach to quantitative solutions. I - Fully saturated problems. *Proc. R. Soc. London*, **A429**, 285 – 309.

Zienkiewicz, O.C., Y. M. Xie, B. A. Schrefler, A. Ledesma and N. Bicânic (1990b). Static and dynamic behaviour of soils: a rational approach to quantitative solutions. II - Semi - saturated problems. *Proc. R. Soc. London*, **A429**, 311 – 321.