Thessaloniki-GREECE 2007

4th International Conference on Earthquake Geotechnical Engineering
June 25-28, 2007
Paper No. 1753

ONE DIMENSIONAL WAVE PROPAGATION CONSIDERING PORE PRESSURE DEVELOPMENT AND DISSIPATION

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ABSTRACT

Pore water pressure generation in saturated sand skeletons during cyclic loading is a very characteristic phenomenon in undrained conditions. The authors found that the empirical, energy based model for pore pressure generation, proposed by Berrill and Davis (1985), reasonably fits data from cyclic laboratory tests.

A computer program for one dimensional site response, incorporating pore pressure generation based on the energy model, and dissipation by one dimensional flow, has been developed. The program uses step by step integration in order to operate a full, nonlinear analysis which takes account of pore water pressure generation and associated stiffness and strength degradation during earthquake loading. The dependence of pore pressure development and dissipation on soil permeability and their significance are illustrated in the paper.

Keywords: Site response, Earthquake , Pore pressure generation, Saturated sand behavior

INTRODUCTION

Saturated sand behavior during cyclic loading is usually characterized by pore water pressure generation. This behavior is related to the tendency of a loose sand skeleton to compress its volume during drained shear. When the sand skeleton is saturated, pore water tends to drain from the sand structure. However, during an earthquake the water may not be able to freely exit the sand structure, resulting in undrained or partially undrained conditions and pore water pressure generation.

PORE WATER PRESSURE GENERATION MODELS

There are two main approaches for prediction of pore water pressure build up:

a. Models which are based on drained conditions.
b. Models which are based on undrained conditions.

The first approach is based on the superposition of volumetric strains, which, in undrained conditions, has to be zero:

$$\varepsilon_v^t = \varepsilon_v^{iso} + \varepsilon_v^{dev} = 0 \quad (1)$$

$\varepsilon_v^t, \varepsilon_v^{iso}, \varepsilon_v^{dev}$ are the volumetric strains due to total, isotropic and deviatoric loading respectively.

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For this purpose empirical models (e.g. Martin et. al. 1975; Byrne, 1991) or other constitutive models (e.g. Frydman, 1972; Bardet, 1986, 1990; Byrne and Mclntyre, 1995; Manzary and Dafalias, 1997; Li and Dafalias, 2000, 2002; Elgamal et al. 2002, Dafalias and Manzary, 2004) can be used to predict pore water pressure generation during cyclic loading. Elgamal et al. (2002) provided a larger list of constitutive models which have been developed to simulate cyclic mobility and/or flow-liquefaction soil response.

The second approach estimates the pore water pressure generation in undrained conditions empirically. Towhata and Ishihara (1985) conducted a series of cyclic undrained tests on loose sand under various loading schemes (torsional, triaxial and a simultaneous combination of them) and came to the conclusion that "*the excess pore pressure development depends solely on the current stress state and the accumulated shear work*". Other researchers (e.g. Nemat-Nasser and Shokooh, 1979; Mosthagel and Habibaghi, 1979; Davis and Berrill, 1982, 1998, 2001; Berrill and Davis, 1985; Yamazaki, Towata and Ishihara, 1985; Law, Cao and He, 1990) also related pore pressure development to shear work or accumulated energy. The advantages of these energy based models are in their simplicity and their capability to fit any kind of loading.

The authors tested some of the above models with triaxial and simple shear tests results obtained by Arulmori et al, (1992), and found that reasonably good fit to the tests results was obtained with the simple model of Berrill and Davis (1985):

$$R_U = \frac{u}{\sigma'_{v0}} = \alpha \cdot \sqrt{\frac{E}{\sigma'_{v0}}} \le 1 \qquad (2)$$

u – accumulated excess pore water pressure.
E – accumulated energy.
σ'_{v0} - initial vertical effective stress
α - calibration parameter.

The model was tested for 6 cyclic simple shear tests and 10 cyclic triaxial tests on Nevada sand at a relative density, DR = 40%, under different initial vertical effective stresses and gave good compatibilities using the same calibration parameter ($\alpha = 2.0$). For a relative density, Dr = 60%, 6 cyclic simple shear tests and 14 cyclic triaxial tests showed good compatibility for a calibration parameter, $\alpha = 1.2$. Figure 1 shows an example of the compatibility in a cyclic simple shear tests on Nevada sand with relative density , Dr = 40%.

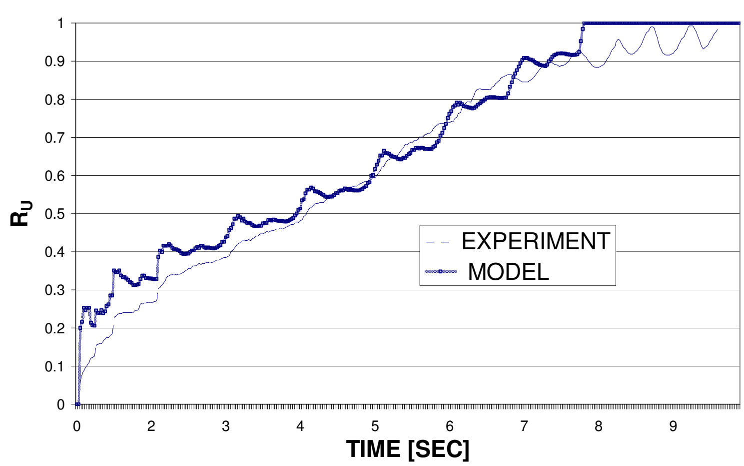

Figure 1 - Normalized pore pressure generation versus time in a cyclic undrained simple shear test on Nevada sand, Dr = 40%

PORE WATER DISSIPATION DURING 1-D WAVE PROPAGATION

The present discussion is limited to a one-dimensional (vertical) model for both propagation of shear waves in the field and flow of water. Since the shear strain profile is not uniform with depth at any time, different excess pore water pressures develop at different depths, resulting in vertical flow. Use of an empirical pore water pressure generation model in undrained conditions, such as the energy based models, enables expression of the pore water pressure change as a superposition of the energy and volume change contributions:

$$\frac{\partial u}{\partial t} = K_{bulk} \cdot \frac{d\varepsilon_v}{dt} + \frac{\partial u}{\partial E}\frac{\partial E}{\partial t} \tag{3}$$

K_{bulk} - bulk modulus.

Assuming incompressibility of water and granular particles, the volumetric strain, $d\varepsilon_v$, is due to flow only, resulting in pore pressure dissipation. Thus, the above equation converts to the following form:

$$\frac{\partial u}{\partial t} = -\frac{1}{\gamma_w} K_{bulk} \cdot (k_{per,x}\frac{\partial^2 u}{\partial x^2} + k_{per,y}\frac{\partial^2 u}{\partial y^2} + k_{per,z}\frac{\partial^2 u}{\partial z^2}) + \frac{\partial u}{\partial E}\frac{\partial E}{\partial t} \tag{4}$$

and, assuming one dimensional flow, eqn (4) reduces to:

$$\frac{\partial u}{\partial t} = -\frac{1}{\gamma_w} K_{bulk} \cdot (k_{per,z}\frac{\partial^2 u}{\partial z^2}) + \frac{\partial u}{\partial E}\frac{\partial E}{\partial t} \tag{5}$$

Thus, the pore water pressure evolution during an earthquake, according to equation (5), can be resolved into two components:

1. Generation of pore water pressure as a function of the accumulated energy.
2. Dissipation of pore water pressure due to flow as a function of the permeability and the bulk modulus of the soil structure.

Eqn (5) is a one dimensional, non-linear differential equation.

NON-LINEAR APPROACH IN THE TIME DOMAIN FOR SITE RESPONSE ANALYSIS

Plasticity-based models have been, and are being used for site response analysis. For example, Elgamal et al. (2002) used a finite element procedure incorporating a plasticity-based constitutive model for site response study, including the effects of pore pressure generation and dissipation. The constitutive model aimed at simulating the cyclic mobility and/or flow-liquefaction phenomena commonly observed in laboratory triaxial and simple shear tests performed on saturated sands. Parameters of the model were obtained by calibration against drained, monotonic triaxial tests, undrained, cyclic triaxial tests, and dynamic centrifuge model tests. These models are often complex, and evaluation of the model parameters not simple. Consequently, the present paper presents an alternative approach, aimed at accounting for the effects of pore pressure development and dissipation on the basis of a simple, empirical, energy-based model incorporated into a one dimensional wave propagation code.

A procedure to solve nonlinear, one dimensional ground response using step by step integration was proposed by Joyner and Chen (1975) who employed the hyperbolic stress-strain model to describe one dimensional shearing. Shiran (2000) extended Joyner and Chen's program and enabled the use of the R-O model (Ramberg and Osgood, 1943) and the logarithmic model (Puzrin and Burland, 1996) within the same framework. None of these approaches considered pore water pressure generation or dissipation.

Site response program

A computer program for one dimensional site response, which incorporates pore pressure generation and dissipation by one dimensional flow, has been developed. The program is formulated using the finite difference approach, by combining the procedure proposed by Joyner (1977) with the numerical, explicit formulation of equation (5), applying the energy based model of Berrill and Davis (1985). The program has the ability to use either the hyperbolic, logarithmic (Puzrin and Burland, 1996) or Hooke stress strain models for virgin loading. Masing rules are adopted for cyclic loading. To account for pore water pressure generation, the program updates the maximum shear modulus according equation (6). Mohr - Coulomb effective stress criterion is used as a failure criterion.

$$G_{\max} = k_g \cdot p_a \sqrt{\frac{\sigma'_{v0} - u}{p_a}} \tag{6}$$

$G_{\max}$ - maximum shear modulus.

k_g - calibration parameter.

p_a - atmospheric pressure.

The program uses the input motion and finds the shear stresses and strains at each differential element of the soil at any specific time. The energies computed according to the stresses and strains are then used to obtain the pore water pressure by the numerical formulation of equation (5) at every element of soil. After obtaining the pore water pressure at each element of soil, the maximum shear modulus and the failure shear stress are updated and the program progress one step in time.

Effect of pore pressure development on site effect

The acceleration time history recorded on rock during the 1987 Whittier earthquake (Fig. 2) was used as input for site response analyses in 4 case studies:

1. Excitation at the base of a sand profile without consideration of pore water pressure generation.
2. Excitation at the base of a sand profile, considering pore water pressure generation, but without consideration of dissipation.
3. Excitation at the base of a sand profile, considering pore water pressure generation and dissipation, assuming a sand permeability of $k_{per} = 10^{-5}$ m/sec.
4. Excitation at the base of a sand profile, considering pore water pressure generation and dissipation, assuming a sand permeability of $k_{per} = 10^{-7}$ m/sec.

The maximum absolute acceleration value of the recorded (input) earthquake was 0.17g.

The acceleration history shown in figure 2 was used as the base input excitation at a site consisting of 16 meters of sand, with water table at a depth of 1 m. The response was studied at a depth of 5 m.

Case 1:

When pore water pressure generation is not considered, the computed acceleration time history at a depth of 5 m. is shown in figure 3. It can be seen that there is a significant amplification; the maximum computed acceleration is 0.54g.

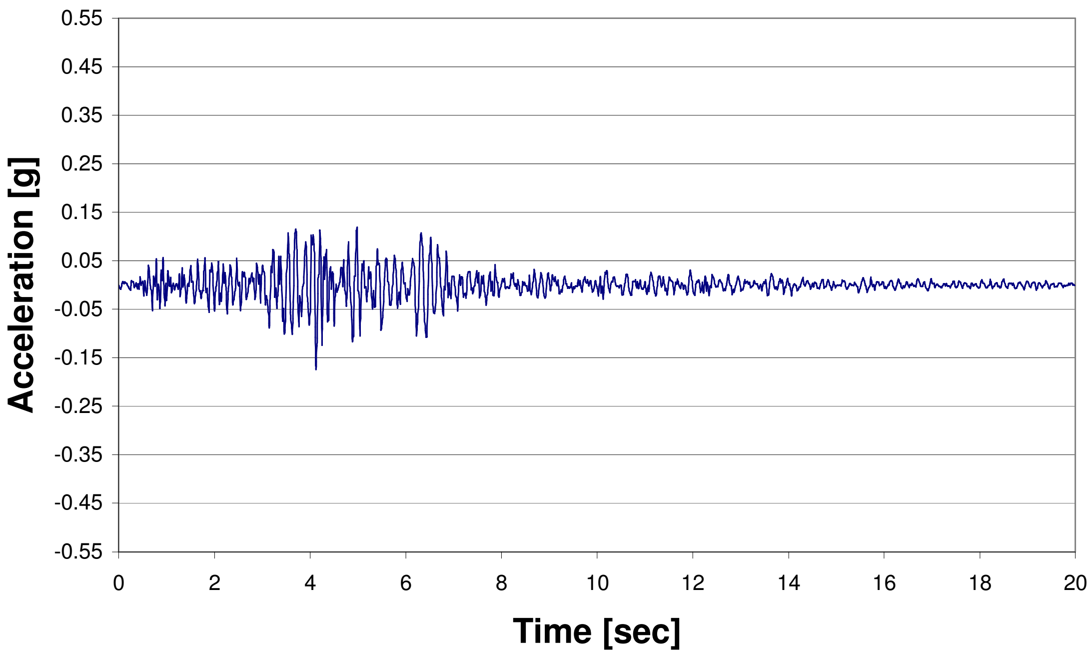

Figure 2 – Recorded acceleration time history recorded on rock during Whittier earthquake.

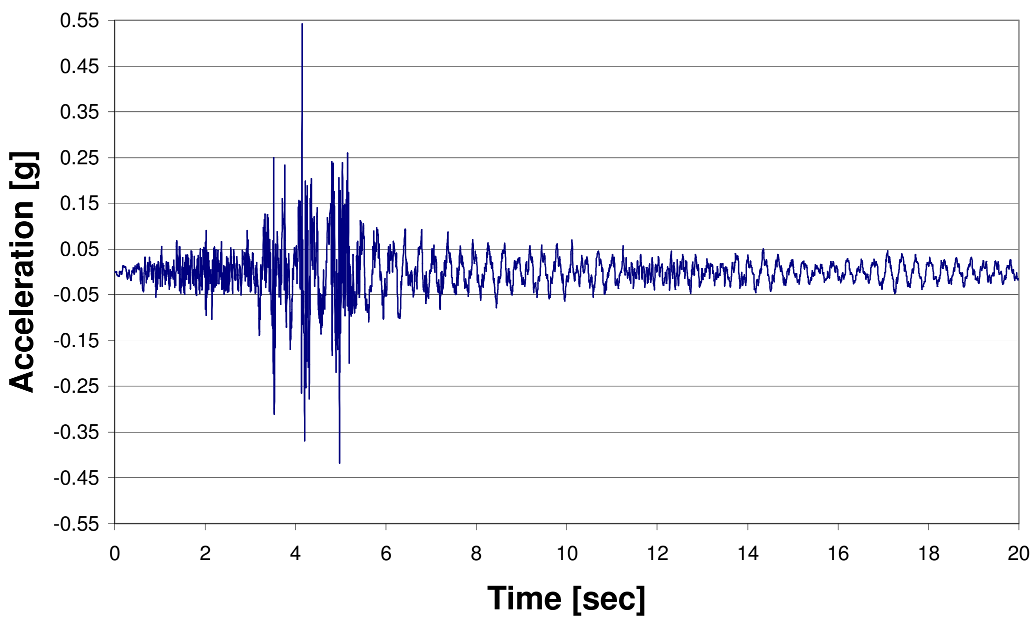

Figure 3 – Computed acceleration time history at 5 m depth in a sand profile, without consideration of pore water pressure generation

Case 2.
Considering pore water pressure generation without dissipation, the resultant acceleration and pore water pressure generation histories during the excitation are depicted in figures 4 and 5 respectively.

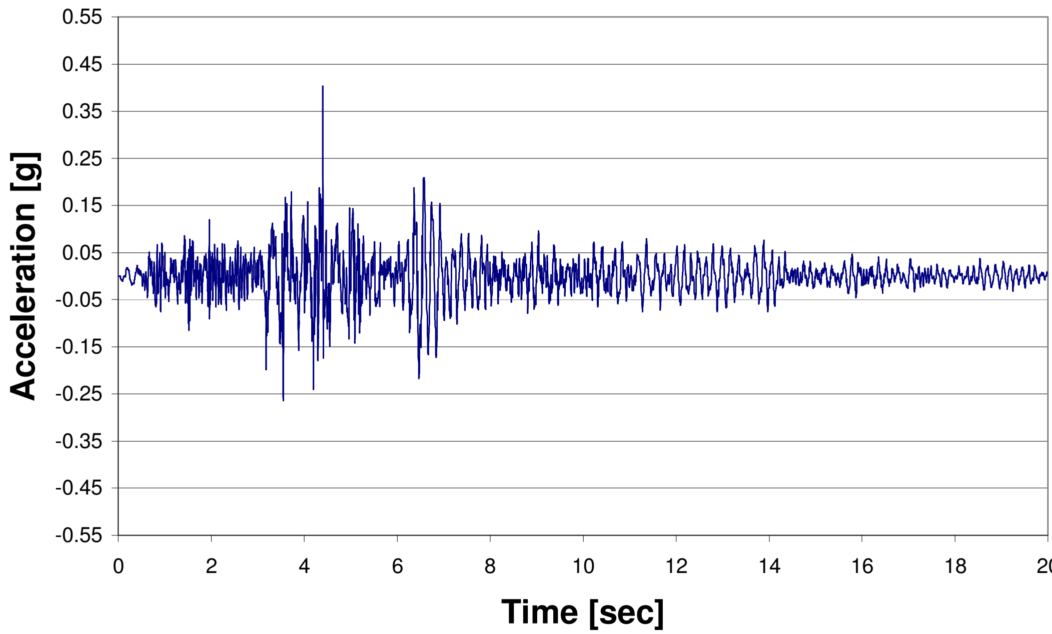

Figure 4 – Computed acceleration time history at 5 m depth in a sand profile, considering pore water pressure generation.

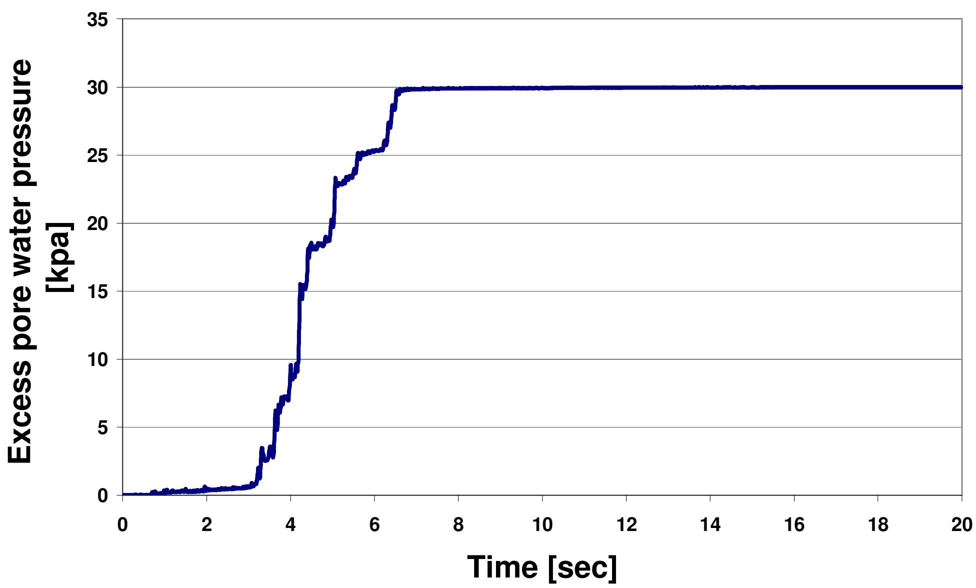

Figure 5 – Computed pore water pressure generation at 5m depth during the earthquake with undrained conditions.

It can be seen that pore water pressure generation influences the acceleration time history. The maximum computed acceleration is 0.41g, significantly less than the value of 0.54g obtained in case 1, when no pore pressure generation was considered. This observation is due to the fact that the stiffness and the strength of the soil have been decreased due to pore water pressure generation. The fact that pore pressure dissipation due to flow has not been considered results in significant pore water pressure build up to a maximum value of 30 kPa, and reduction of 30% in the stiffness of the sand (see figure 6). It can be seen that after 7 seconds there is no more pore water pressure build up.

This is due to the fact that at this time the accelerations are very small resulting in small strains resulting in insignificant energy accumulation.

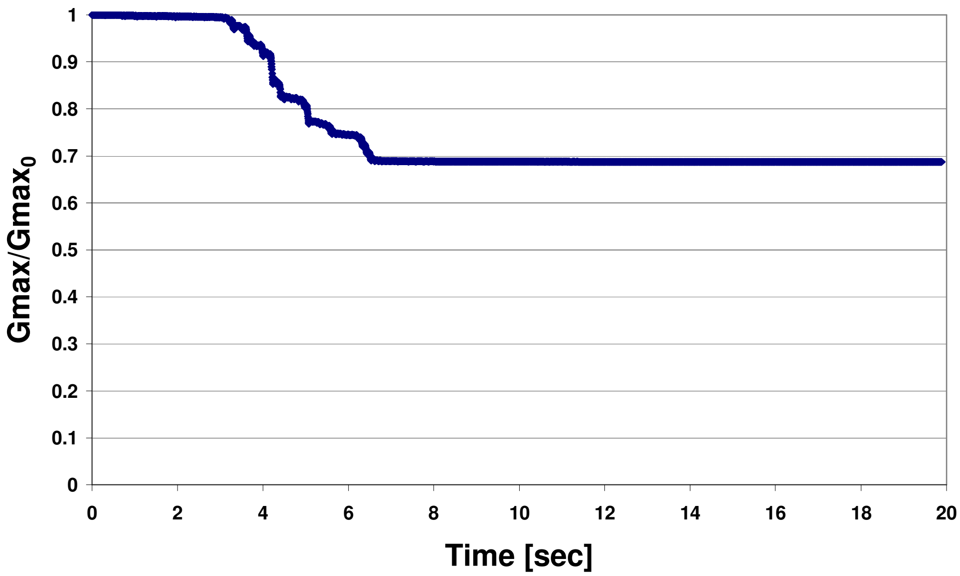

Figure 6 –Stiffness degradation during undrained conditions.

<u>Case 3</u>

When pore water pressure generation and dissipation (via flow) are considered, and the permeability of the sand is taken as 10^{-5} m/sec, the resultant acceleration time history, pore water pressure generation and stiffness degradation at 5 m depth, are depicted in figures 7-9 respectively.

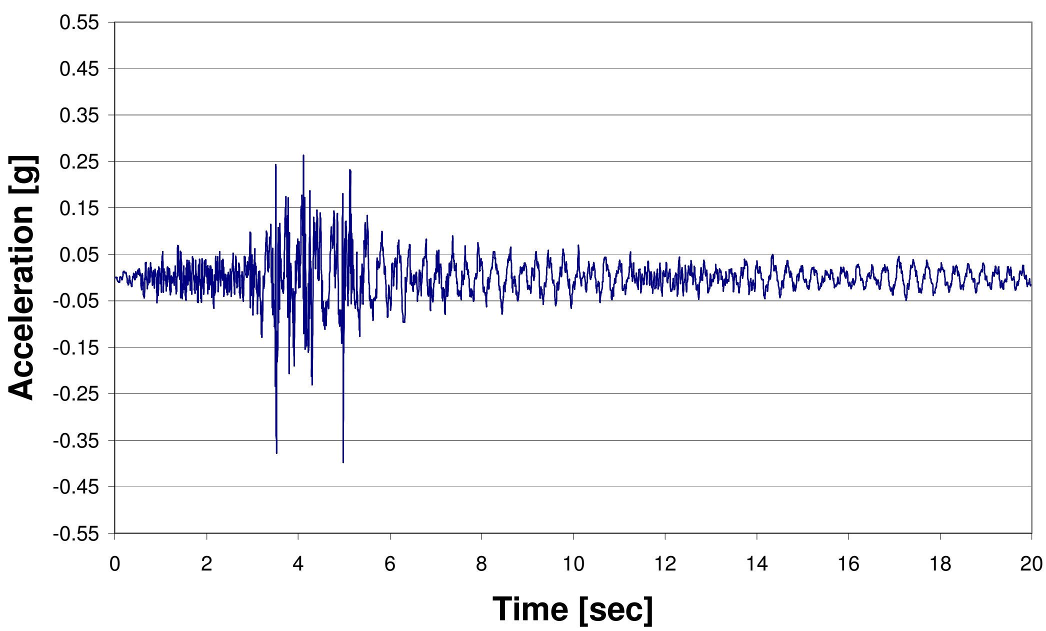

Figure 7 – Computed acceleration time history at 5 m depth in a sand profile considering pore water pressure generation and dissipation.

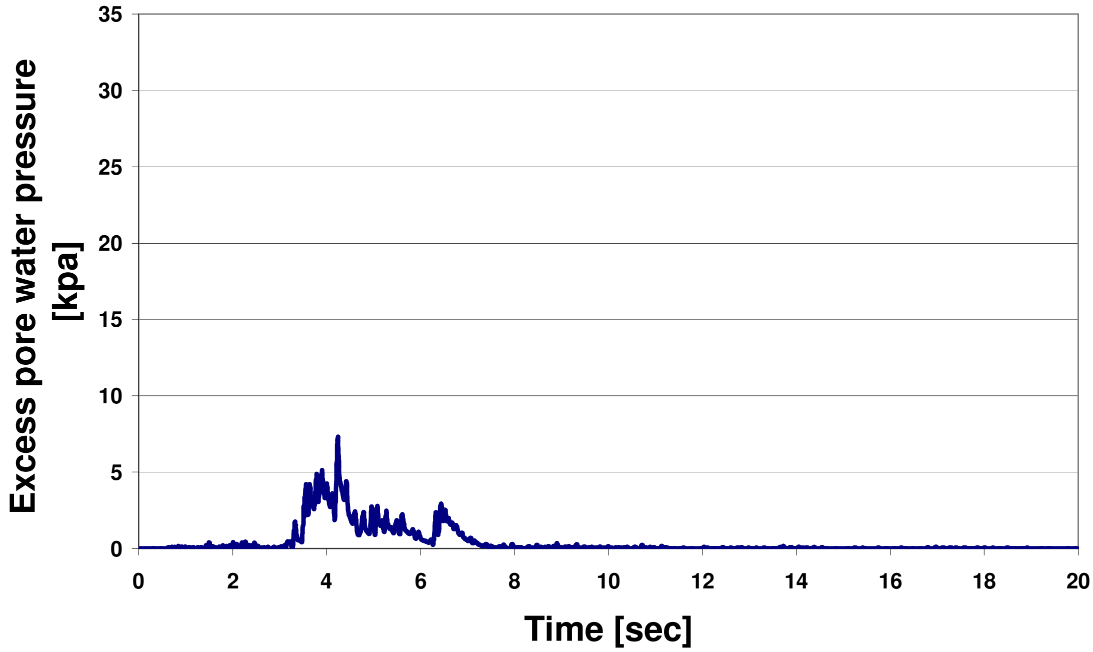

Figure 8 – Computed pore water pressure generation at 5 m depth during the earthquake with pore pressure generation and dissipation.

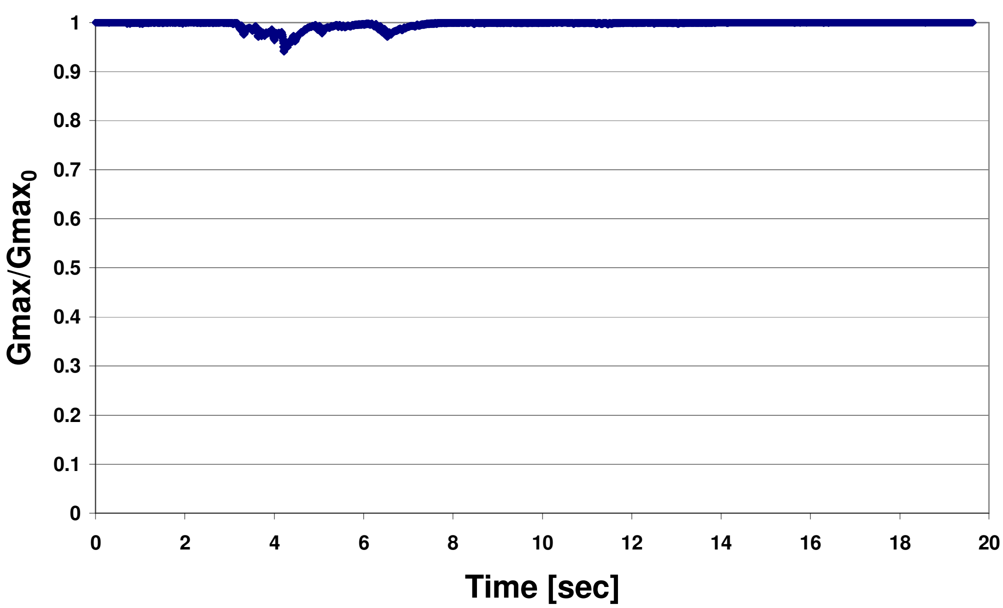

Figure 9 –Stiffness degradation at 5 m depth during pore pressure generation and dissipation.

It can be seen that as a result of dissipation of pore water pressure, the pore pressure generation is significantly decreased (maximum value of 7 kPa). Almost no degradation of stiffness occurs.

Case 4

The conditions of case 3 were reanalyzed for a sand permeability of 10^{-7} m/sec. Figure 10 shows the pore water pressure history for the three permeability cases studied:

$k_{per} = 10^{-5}$ m/sec, $k_{per} = 0$, and $k_{per} = 10^{-7}$ m/sec.

It can be seen that even with the lower permeability of $k_{per} = 10^{-7}$ [m/sec], the maximum pore water pressure developed is close to, but less than in the case of impermeable conditions.

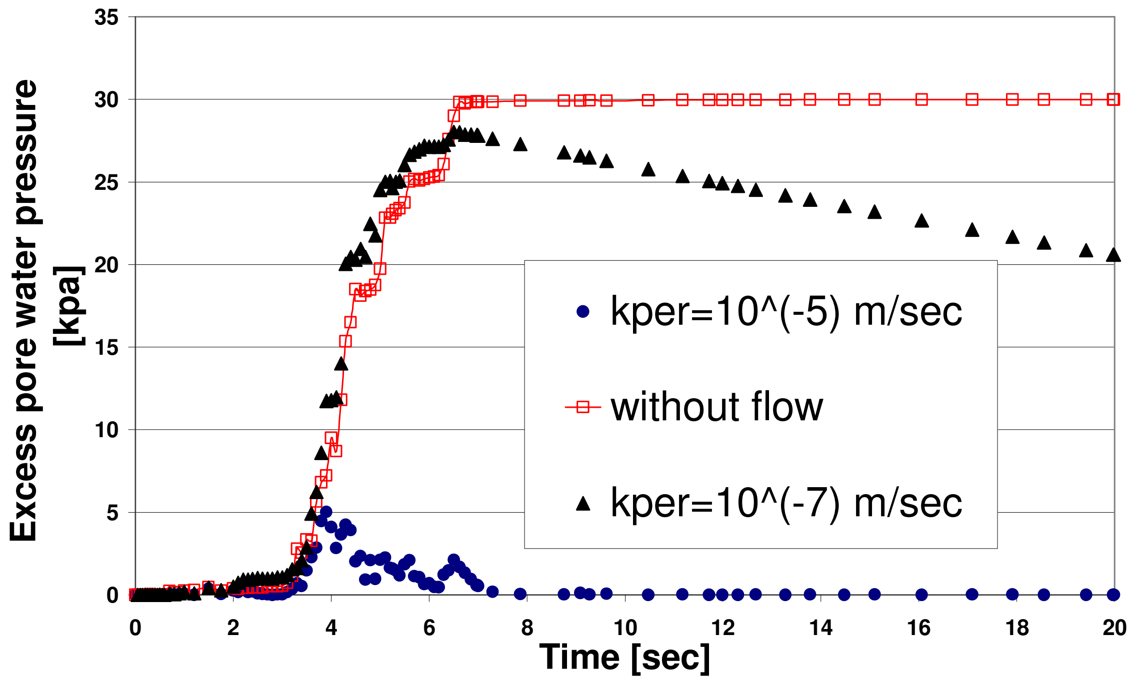

Figure 10 – Comparison of pore pressure generation for different soil permeabilities

CONCLUSIONS

It has been found that the energy based model for pore pressure generation, proposed by Berrill and Davis (1985) reasonably fits data from laboratory cyclic undrained shear tests.
A computer program for one dimensional site response, incorporating pore pressure generation based on the above energy model, and dissipation by one dimensional flow, has been developed. As an example, a recorded earthquake was used as input for the program, and four cases were studied in order to illustrate the influence of the permeability on the computed acceleration near the soil surface during earthquake excitation. It was shown that for high permeability, in a shallow deposit of saturated sand on bedrock, the influence of pore pressure generation on the accelerations and stiffness degradations is insignificant as a result of rapid dissipation. However, pore pressure generation in saturated sand deposits of low permeability (e.g silty sands) can significantly influence the acceleration time history, and the degradations of stiffness and strength during the earthquake.

ACKNOWLEDGEMENTS

The research presented in this paper is being funded by the Israel Ministry of Housing and Construction, through the National Building Research Institute at the Technion.

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