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Shaking table test and 2D nonlinear analysis for sand-structure system

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ABSTRACT: Verification of a two dimensional nonlinear analysis is discussed through comparisons of results of a shaking table test for a one - story structure standing on a dry or a saturated sandy deposit. The proposed nonlinear method can well represent the observed nonlinear response of the dry or the saturated sandy deposit including the structure under the condition of shear strain of 10^{-3}.

1. INTRODUCTION

The phenomena of site amplification and soil - structure interaction are affected by the nonlinear behavior of soil including liquefaction when high intensity ground motions are caused by earthquakes. The dynamic nonlinear response of soil can be calculated with step by step integration techniques, which require relevant computational efforts. Although these methods are considered to be the most precise at present, there have been a few studies in which their effectiveness for evaluating the nonlinear response of the soil - structure system has been confirmed by comparing their numerical results with observed results. This is due to the insufficient measured data for nonlinear soil - structure systems: a few valuable results were obtained from in situ forced vibration tests (Vaughan 1983) , shaking table tests (Wakui 1990, Fukutake 1990, Ohtsuki 1992) and earthquake observations (Lacy 1987). On the other hand, centrifuge tests have been recently conducted to study the nonlinear soil - structure interaction (Finn 1985, Scott 1990). The centrifuge test has become an effective experiment for examining the geotechnical problems including structures under the high nonlinearity of soil condition.

In the present paper, a series of shaking table tests is conducted for investigating the nonlinear response of a dry or a saturated sandy deposit, on which a one - story structure stands. The verification of the proposed analysis is also discussed through the comparisons of the experimental data under the condition of shear strain in the 10^{-3}.

2. 2 DIMENSIONAL NONLINEAR METHOD

In the authors' previous papers (Ohtsuki 1987, Fukutake 1990), an effective stress method was proposed for the analysis of liquefaction explicit - implicit finite element method, in which an explicit FEM was applied to the ground and an implicit FEM was used for the structure. In the present paper, the implicit method with the Newton - Raphson iteration is applied to simulate the response of the soil - structure system under the condition of a shear strain of about 10^{-3} because the method can maintain control on the accuracy of nonlinear equations. The basic equations to be solved in nonlinear analysis are

$$\{{}^{t+\Delta t}\overline{R}(U)\} - \{{}^{t+\Delta t}F(U)\} = 0 \qquad (1)$$

and

$$\{{}^{t+\Delta t}\overline{R}(U)\} = \{{}^{t+\Delta t}R\} - [M]\cdot\{{}^{t+\Delta t}\ddot{U}\} - [C]\cdot\{{}^{t+\Delta t}\dot{U}\} \qquad (2)$$

where $\{{}^{t+\Delta t}R\}$ is a vector of external nodal point loads corresponding to time $t+\Delta t$; $\{{}^{t+\Delta t}F\}$ is a vector of internal applied nodal point forces; [M] is a mass matrix; [C] is a damping matrix; and $\{{}^{t+\Delta t}\ddot{U}\}$ and $\{{}^{t+\Delta t}\dot{U}\}$ are vectors of nodal accelerations and velocities. The increment of the strain is written as:

$$\{d\varepsilon\} = [B]\cdot\{dU\} \qquad (3)$$

where [B] is a strain - displacement matrix and { dU } is a vector of incremental displacements between time t and time $t+\Delta t$. Considering the effect of the bulk modulus for pore water on the governing equation for granular soil to satisfy the undrained condition, the increment of the total stress $\{d\sigma\}$ is obtained from the equation

$$\{ d\sigma \} = \left[[D] + \{m\}\frac{K_w}{n_e}\{m\}^T \right] \cdot \{ d\epsilon \} \tag{4}$$

where [D] denotes a stress - strain matrix for soil. Matrix [D] is updated by the current effective stress at every time step obtained from the hybrid constitutive relation of the modified Ramberg - Osgood model (R - O model), which defines the stress - strain relationship, and the Bowl model (Fukutake 1989) which assesses the excess pore water pressure. K_w / n_e corresponds to the ratio of the fluid bulk modules to the porosity and $\{ m \} = \{ 1,1,0 \}^T$. Thus, the internal force $\{ {}^{t+\Delta t}F \}$ is written as :

$$\{ {}^{t+\Delta t}F \} = \int_v [B]^T \cdot \{\sigma^{t+\Delta t}\} \, d(vol) \tag{5}$$

where

$$\{\sigma^{t+\Delta t}\} = \{\sigma^t\} + \{d\sigma\} \tag{6}$$

3. CONSTITUTIV MODEL

The relationship between shear stress and shear strain is defined by the R - O model , and the relationship between shear strain and volumetric strain is modeled by the new dilatancy model, which we call a Bowl model. The Bowl model is summarized as follows. The resultant shear strain Γ, which represents the radial distance from the origin, and the cumulative shear strain G^*, which represents the length along the shear strain path, are defined as follows :

$$\Gamma = \sqrt{\gamma_{xy}^2 + \gamma_{yz}^2} \tag{7}$$

$$G^* = \sum \sqrt{\Delta\gamma_{xy}^2 + \Delta\gamma_{yz}^2} \tag{8}$$

where γ_{xy} ($\Delta\gamma_{xy}$) and γ_{yz} ($\Delta\gamma_{yz}$) are the shear strains (shear strain increments) in the x- and z-directions on the horizontal plane, respectively. It is found that a soil particle climbs up and slips down on other particles while a soil sample is compressed during the multi - directional simple shearing. In order to represent this phenomena, it is assumed that the average soil particle moves on the bowl - like surface, while the bowl itself is compressed along the axis of the volumetric strain. Since the bowl - like surface is assumed to be represented by the body of revolution, the bowl shape is a function of the resultant shear strain Γ. Thus the average soil particle moving on the bowl surface causes the normal strain ϵ_Γ which is defined as follows :

$$\epsilon_\Gamma = A \cdot \Gamma^B \quad \text{(positive dilatancy)} \tag{9}$$

where A and B are soil parameters. It is assumed that the bowl itself is compressed by the shearing disturbance and G^* can be chosen for the index of the disturbance. Thus the bowl transferring along the axis of the volumetric strain produces the normal strain ϵ_G as follows :

$$\epsilon_G = \frac{G^*}{C + D \cdot G^*} \quad \text{(negative dilatancy)} \tag{10}$$

where C and D are also soil parameters. The volumetric strain ϵ^s_v due to the shearing is obtained from a following equation, which represents the superposition of the positive dilatancy and the negative dilatancy.

$$\epsilon_v^s = \epsilon_\Gamma + \epsilon_G \tag{11}$$

The circular elastic zone is considered in order to suppress the dilatancy under the small amplitude of the shear strain. The radius of the elastic zone is determined by the lower limit of the liquefaction resistance X_l. The incremental normal strain $d\epsilon_G$ is not produced within the elastic zone. Considering the one - dimensional swelling component ϵ^c_v, the total volumetric strain ϵ_v is obtained from a following equation:

$$\epsilon_v = \epsilon_v^s + \epsilon_v^c \ , \ \epsilon_v^c = \frac{C_s}{1+e_0} \log\frac{\sigma'_m}{\sigma'_{mo}} \tag{12}$$

where e_0 and C_s are an initial void ratio and a swelling index, respectively. σ'_{m0} is an initial effective mean stress. The excess pore water pressure is obtained from the condition of no volumetric strain increment under the undrained condition. In the present analyses, the x - directional simple shearing is only considered so that the values of γ_{yz} and $\Delta\gamma_{yz}$ shown in equations (7) and (8) are equal to zero. The R - O model and Bowl model are extended to the two - dimensional problems, considering the uncoupled relationship between the shear stress - strain and the normal stress - strain (Fukutake 1990).

4. EXPERIMENTAL MODEL

A series of shaking table tests was conducted for investigating the nonlinear response of a dry or a saturated sandy deposit on which a one - story structure stands. As shown in Fig.1, the model ground is made of Toyoura sand having a depth of 98 cm, a length of 200 cm and a width of 150 cm. The ground container is made from a stack of 18 aluminum rectangular rings, each ring being 5 cm high and 3 cm wide. Ball bearings are installed between the aluminium rings to reduce the shear friction. The one - story structure is made of steel columns, plates, and a viscous damper using asphalt. A natural frequency and a damping factor of the structure are 15.1 Hz and 4 per cent, respectively. As illustrated in Fig.1, the accelerations at different locations in the ground and the structure are measured by

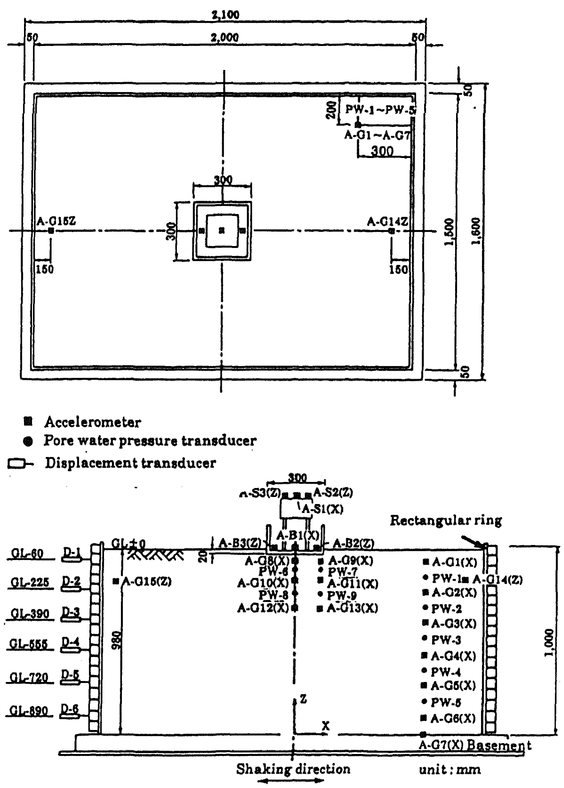

Figure 1. Experimental model

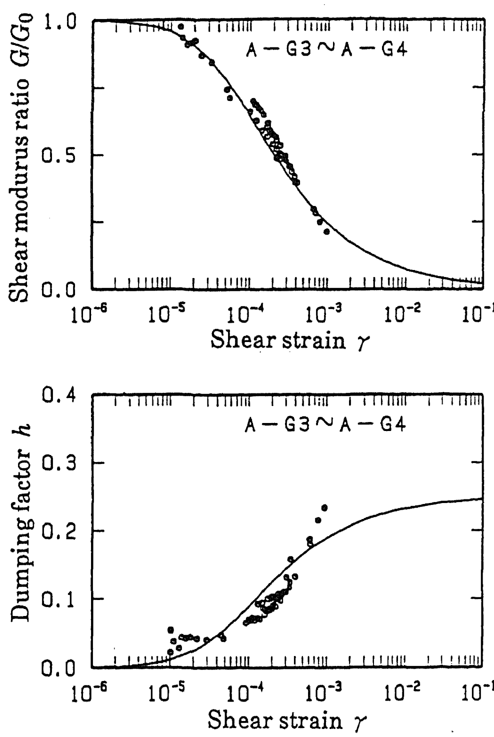

Figure 2. Relationship of $G \sim \gamma$ and $h \sim \gamma$ for layer between (A − G3) and (A − G4)

accelerometers (A-G1~A-G8). Displacement transducers (D-1~D-5) are attached to the lateral wall of the container to measure the horizontal displacement of the ground. Pressure transducers (P-W1~P-W5) are installed in the saturated sandy deposit to measure the excess pore water pressure.

5. RELATIONSHIPS OF G - γ AND h - γ

Since the relative density of the dry sandy deposit is about 98 per cent, the present shaking table test can reproduce the same nonlinear response of the ground due to the same amplitude of the input motion. Thus, the relationships of G - γ and h - γ are obtained directly from the resonance curve of the dry sandy deposit for the sinusoidal waves by the back analysis (Matuda 1986). The ground is modeled by the multi - degree of freedom system featuring shear springs and lumped masses. As a result, the shear modulus and the damping factor for each layer are obtained from the calculated complex stiffness for the above model. For instance, the values of the shear modulus and the damping factor for the layer between A-G3 and A-G4 are shown by circular plots in Fig. 2, together with the solid lines indicated as the mean curves for the relationships of G - γ and h - γ. Those lines are determined to represent the nonlinear relationships of G - γ well under the condition of the shear strain ranging from 10^{-5} to 10^{-3}. An initial shear modulus, G_0, an initial reference shear strain, $\gamma_{0.5}$ and a maximum damping factor, h_{max} are read off from the curves of G - γ and h - γ.

6. SIMULATION OF RESPONSE OF SANDY DEPOSIT WITH STRUCTURE

The two dimensional nonlinear analysis based on the constitutive relation of the R - O model is applied to simulate the response of the structure standing on the dry sandy deposit due to the TAFT earthquake of 250 Gal.

The soil - structure system is modeled by the two dimensional beams and solid elements. The ground is considered to be a plane strain condition having a width of 30 cm, which is equal to the width of the foundation of the structure. Fixed and roller conditions are used in the bottom and lateral boundaries of the analytical model.

The computed and observed Fourier spectra

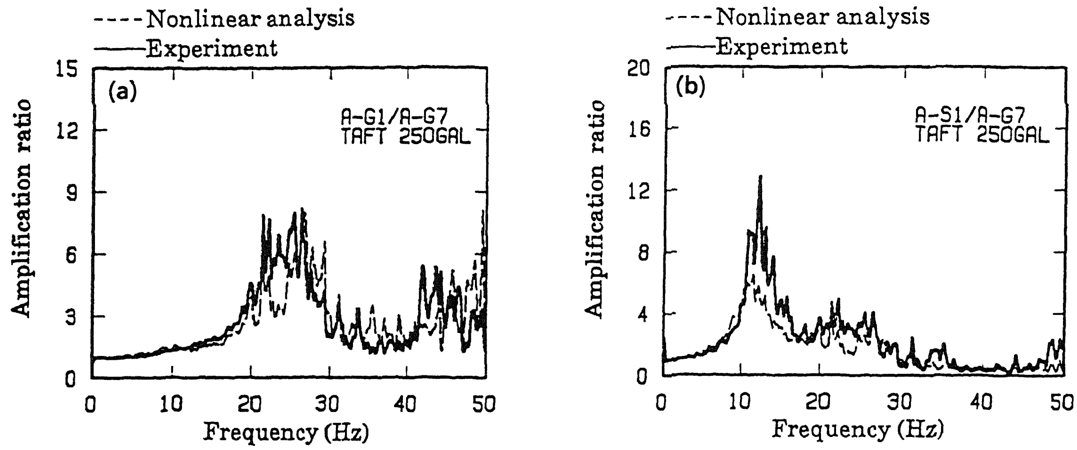

Figure 3. Computed and measured Fourier spectra ratios

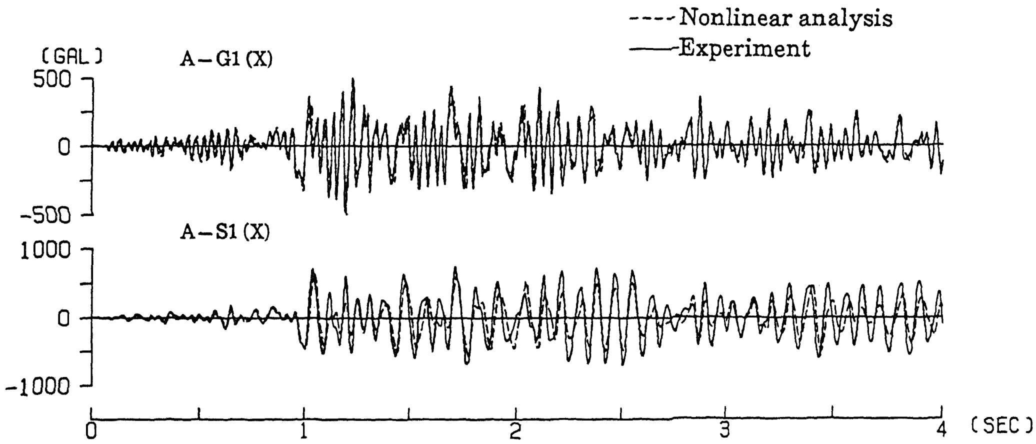

Figure 4. Computed and measured accelerations

ratios between A-G7 and A-G1 and between A-G7 and A-S1 are shown in Figs. 3(a) and (b). The computed Fourier spectra ratios agree with those of the experiment. The component of the computed ground response tends to have two predominant peaks while the observed one has a gently sloping peak. The computed amplification ratio on the top of the structure is smaller than that of the observation at the predominant frequency of around 12 Hz.

Figure 4 shows the computed and observed accelerations on the surface and on the top of the structure. The amplitude and phase of those accelerations obtained from the nonlinear analysis coincide well with the measured accelerations. However, the amplitude of the computed acceleration on the top of the structure is relatively smaller than that of the observed one after two seconds.

Next, the response of the structure standing

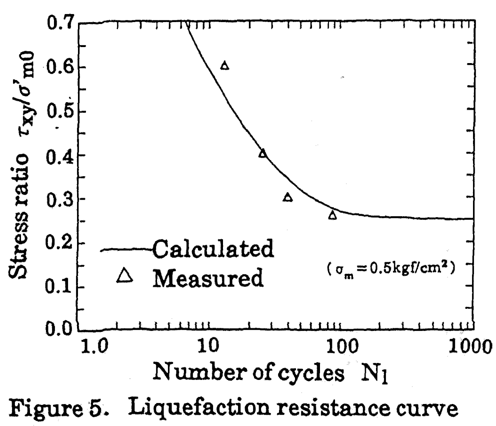

Figure 5. Liquefaction resistance curve

on the saturated sandy deposit is simulated by the nonlinear analysis with the hybrid constitutive relation of R - O model and Bowl

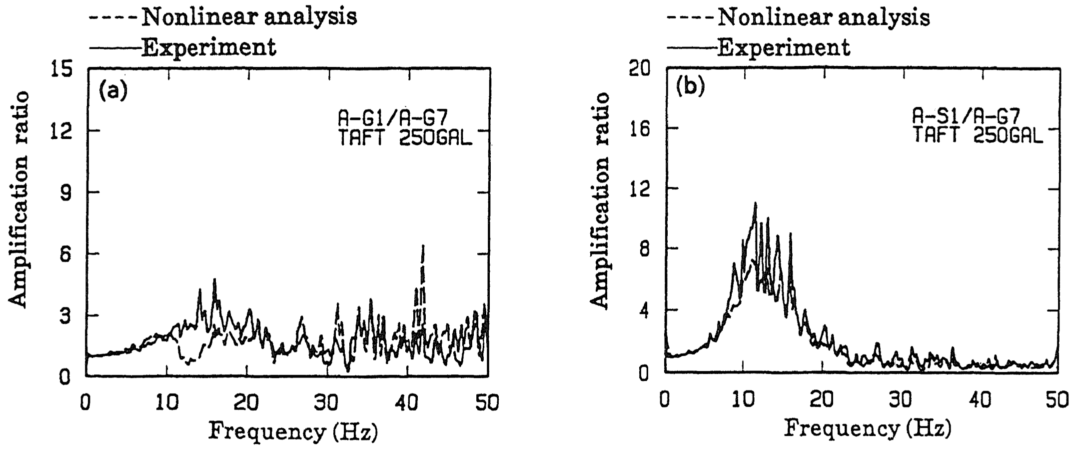

Figure 6. Computed and measured Fourier spectra ratios

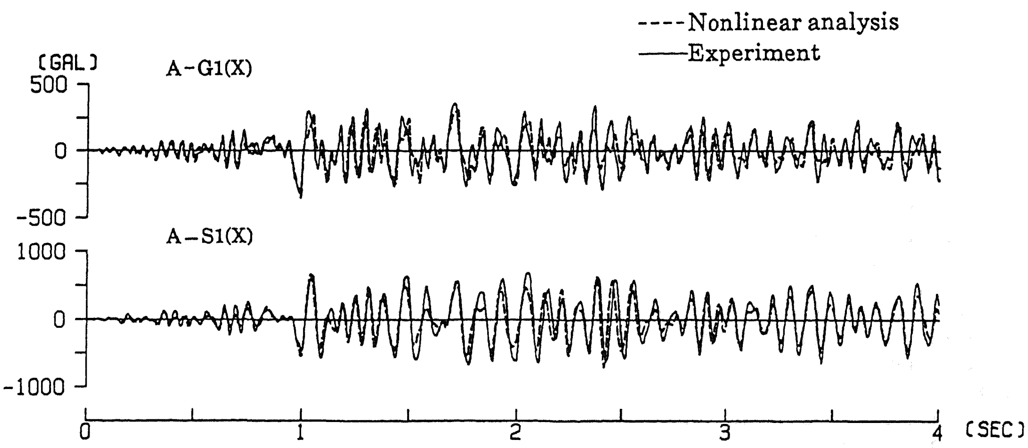

Figure 7. Computed and measured accelerations

model. The same values for the dry sand are also used for the saturated sand. In the Bowl model for assessing the excess pore water pressure, the value of the swelling index $Cs/(1+e_0)$ is obtained from the past data of Toyoura sand. The value of the lower limit of the liquefaction resistance X_l is 0.25, which is read off from the present liquefaction resistance curve of the Toyoura sand having a mean stress of 0.5 kgf/cm^2 (49 kps) as shown in Fig. 5.
Parameters A, B, C and D are selected by identifying the liquefaction resistance curve.
From the figure, the present hybrid constitutive relation represents the experimental results with a reasonable degree of accuracy.

The computed Fourier spectra ratios between A-G7 and A-G1, and between A-G7 and A-S1 are shown in Figs. 6(a) and (b). Although the component of the computed ground response tends to have two predominant peaks as indicated in the calculated results for the dry sandy deposit, the present computed Fourier spectra ratios agree approximately with those of the experiment.

The computed and measured accelerations on the surface of the ground and on the top of the structure are also shown in Figs. 7. The computed accelerations agree well with the observed ones in amplitude and phase.

Figure 8 indicates the measured and computed excess pore water pressures. The positive dilatancy due to the cyclic mobility appears predominantly in the excess pore water pressures observed in the ground beneath the structure (PW-6 and PW-7) compared with that at a distance from the structure (PW-1). This is due to the fact that the response of the structure with its rocking vibration affects the pore water

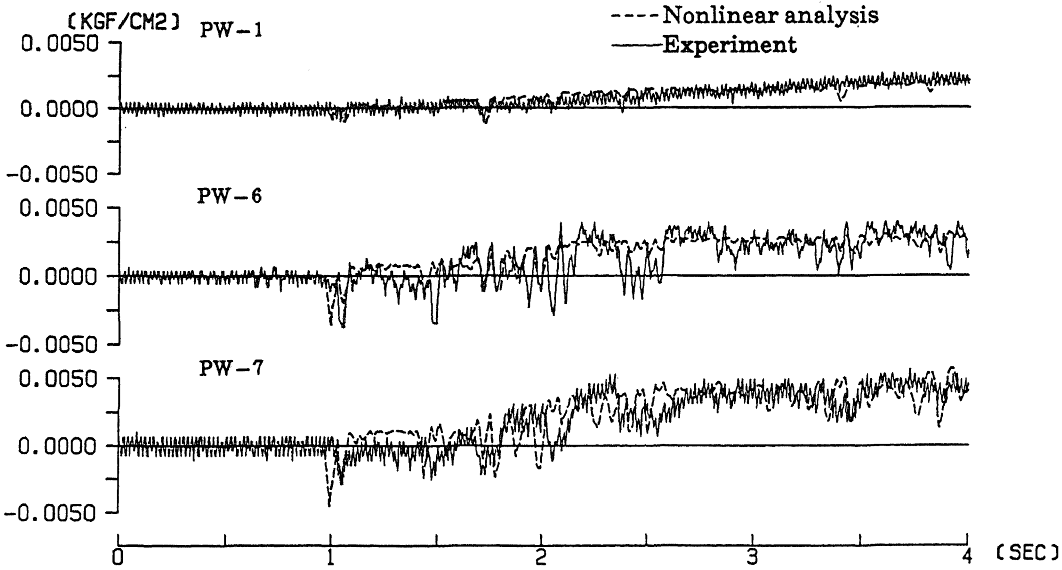

Figure 8. Computed and measured excess pore water pressures

pressure. The computed excess pore water pressures agree approximately well with the results of experiment, although the increment of the positive dilatancy is not observed clearly in the computed excess pore water pressure.

The distribution of the maximum excess pore water pressure is illustrated in Fig. 9. The excess pore water pressure ratio of about 40 per cent of the initial overburden stress appears at the surrounding ground of the structure. Resultantly, the elongation of the predominant period caused by the accumulating excess pore water pressure is not distinguished.

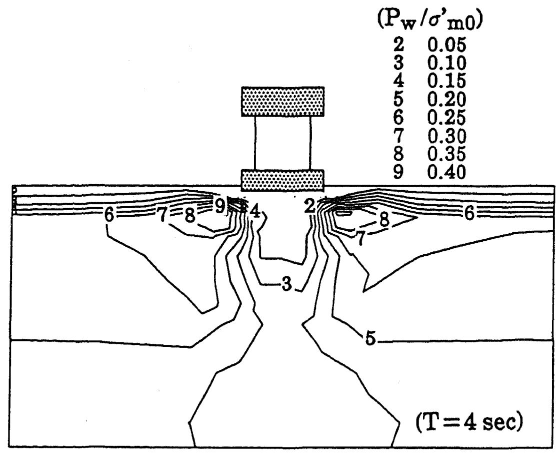

Figure 9. Distribution of computed maximum excess pore water pressure ratio

7. CONCLUSION

The verification of the two dimensional nonlinear analysis is discussed through the comparisons of the experimental data for the structure standing on the dry or the saturated sandy deposit. The proposed nonlinear method can represent well the observed nonlinear response of the dry and the saturated sandy deposit including the structure. The method, however, should be applied carefully to assess the response of the soil - structure system, when the three dimensional interaction affects significantly the response of the model.

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