ASSESSMENT OF THE KOBE PORT ISLAND LIQUEFACTION THROUGH ANALYTICAL SIMULATION OF THE VERTICAL ARRAY RECORDS

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ABSTRACT

An effective stress analysis of the vertical seismic array site at Port Island is conducted in order to assess the characteristics of the ground response induced by the Kobe earthquake. To investigate the response of Masado layer and the effects of the directionality of the motion a series of shallow model analyses are conducted in different directions. Deep model analysis is used to evaluate the response of the alluvial gravelly sand lying from 28 m to 37 m depth.

1. INTRODUCTION

In the January 17, 1995, Hyogoken-Nanbu Earthquake, of magnitude 7.2, widespread liquefaction occurred in the seaside area of Kobe city causing large ground deformation and severe damages to engineering structures. The liquefaction was particularly extensive in the reclaimed lands of Port Island and Rokko Island. Motions induced by the main shock of this quake have been recorded at several sites that exhibited massive liquefaction of reclaimed soil. Among these records, particularly interesting are those obtained with the vertical seismic array at Port Island since they include records at four different depths of the soil profile ranging between the ground surface and 83 meters depth. Location of the vertical seismic array site at Port Island is indicated in Fig. 1.

The objective of this study is, firstly, to simulate the recorded motions at Port Island by means of an effective stress analysis, and then, to use the results of the analysis for clarification of the characteristics of the ground response such as the extent of the liquefaction, development and distribution of excess pore pressures and shear strains etc.

2. SOIL PROFILE

Characteristics of the soil profile at the seismic vertical array site including SPT N-values, shear wave velocities, material types and location of the accelerometers along the depth of the profile are shown in Fig. 2 (Toki, 1995). These properties are obtained by a site investigation conducted prior to the Hyogoken-Nanbu Earthquake. We can distinguish five different layers that comprise the soil deposit up to a depth of about 80 m. The surface layer is a reclaimed Masado, a decomposed granite from the Rokko mountains that was used to form the man-made Port Island. The Masado layer, thick about 18 m, overlies the original sea-bed layer of alluvial clay and an alluvial gravelly sand layer underneath, with thicknesses of 10 and 9 m, respectively. Below these layers lie a diluvial gravelly sand layer and a diluvial clay layer, each with a thickness of about 22 m. The water table is approximated at 3 m below the surface.

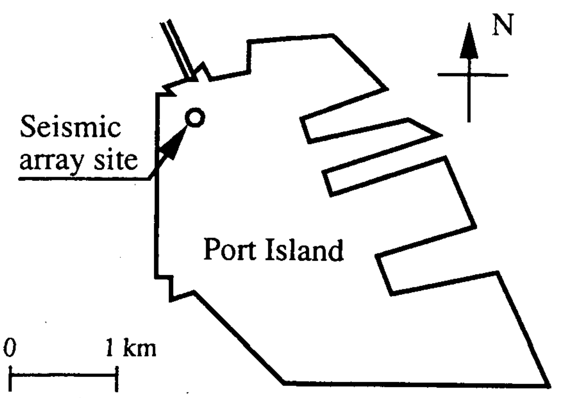

Fig. 1 Vertical seismic array site at Port Island

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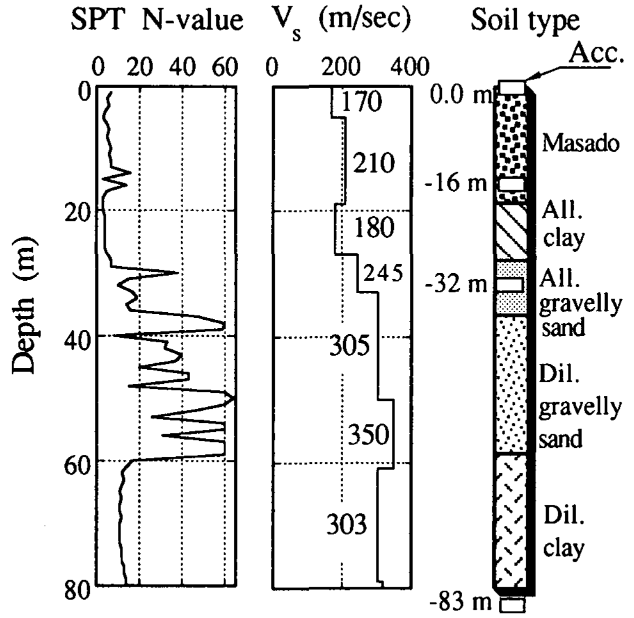

Fig. 2 Soil profile of the vertical seismic array site at Port Island

3. GROUND MOTION RECORDS

The motion induced by the main shock of the Hyogoken-Nanbu Earthquake was successfully recorded with the vertical seismic array operated by the Development Bureau of Kobe City (Toki, 1995). The seismic array consists of 4 sets of accelerometers placed at the ground surface and at depths of 16 m, 32 m and 83 m, as indicated in Fig. 2. Each set has three accelerometers, oriented in N-S, E-W and U-D directions, respectively. The suggestion by Ansary et al. (1995) that there was an orientation error for the horizontal accelerometers at 83 m depth is accepted in this study, and therefore, corrected horizontal records by an anti-clockwise rotation for 19 degrees are used for this depth.

In order to investigate the characteristics of the ground response a simple analysis of the recorded motion is done by comparing the records at two adjacent depths, as shown in Fig. 3. For example, the top part of Fig. 3 shows comparative plots of the N-S records at the ground surface and 16 m depth. The record at the ground surface is shifted on the time axis towards left so as to bring the initial large-amplitude cycles of the two records in phase. The time increment of the shift, Δt, can be considered as an estimation for the upward propagation time of the shear waves between the

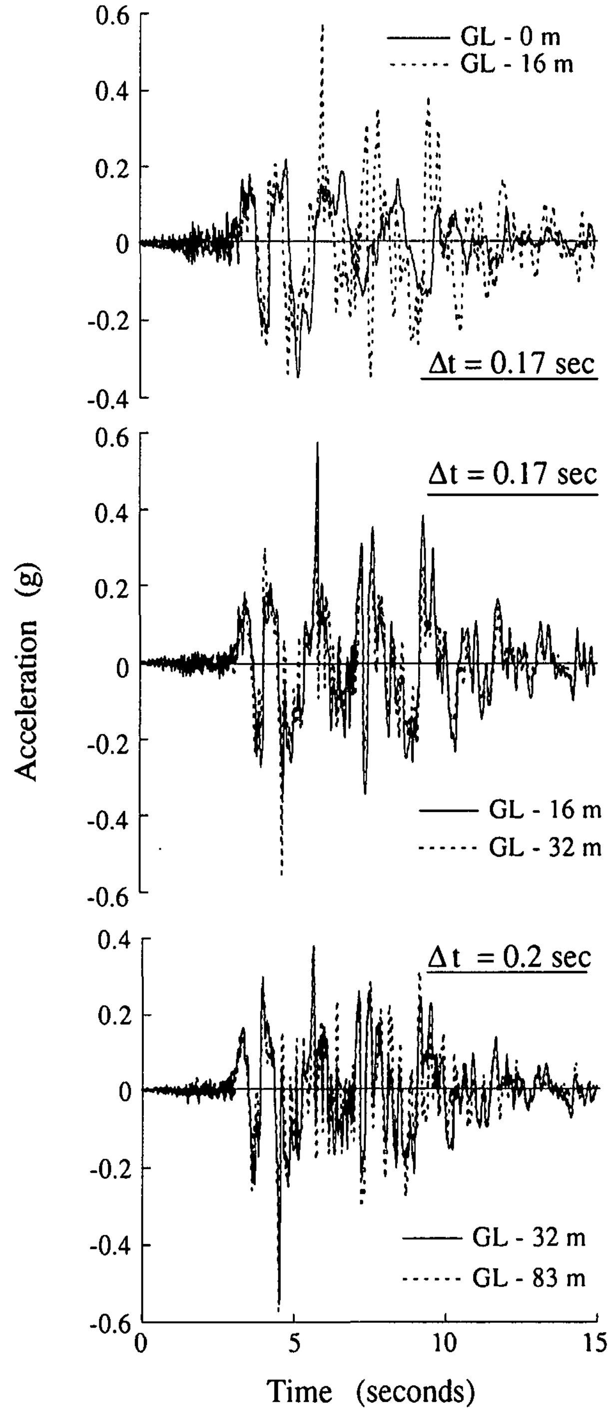

Fig. 3 Comparison between the N-S records at adjacent measuring points

two measuring points. Knowing the distance, ΔH, and the propagation time, Δt, the average shear wave velocity between the two adjacent measuring points is calculated as $V_r = \Delta H / \Delta t$. On the other hand, the average elastic shear wave velocities, V_e, are calculated from the velocity logging data shown in Fig. 2. Finally, the average elastic shear wave velocities, V_e, and those calculated from the records, V_r, are used to estimate the degradation of shear modulus as defined by the ratio G_r / G_e. Following the above procedure, degradation of the shear moduli between each of the adjacent measuring points is calculated and summarized in Table 1.

Table 1. Degradation of shear moduli with depth

Depth (m)	Δt (sec)	V_e (m/sec)	V_r (m/sec)	G_r / G_e
0 - 16	0.17	197	94	0.23
0 - 16*	0.40		40	0.04
16 - 32	0.17	206	94	0.21
32 - 83	0.20	313	255	0.66

* - after liquefaction

It can be seen in Fig. 3 that following the first one and a half large-amplitude cycles, the surface motion drifted towards right indicating that cyclic softening due to liquefaction occurred. It is illustrated in Fig. 4 that a shift on the time axis of 0.40 seconds is needed in order to bring the later part of the records in phase. As indicated in Table 1, following the liquefaction, the shear modulus of Masado was reduced to only 4% of the initial shear modulus G_e.

Unlike the response of the top 16 m as exemplified by the remarkable change of the motion from 16 m depth to the ground surface, the ground response below 16 m is characterized with a remarkable similarity between the motions of any two adjacent recording points. Completely identical conclusions and numerical values as those listed in Table 1 are obtained by implementing the above procedure to the recorded E-W components of the motion.

4. EFFECTIVE STRESS ANALYSIS

A fully coupled effective stress analysis of the Port Island soil profile was conducted using the finite element code DIANA-J with the Stress-Density Model (S-D Model), (Cubrinovski, 1993) as a constitutive model. All the analyses are drained 1-D soil-column analyses for an assumed plane strain condition.

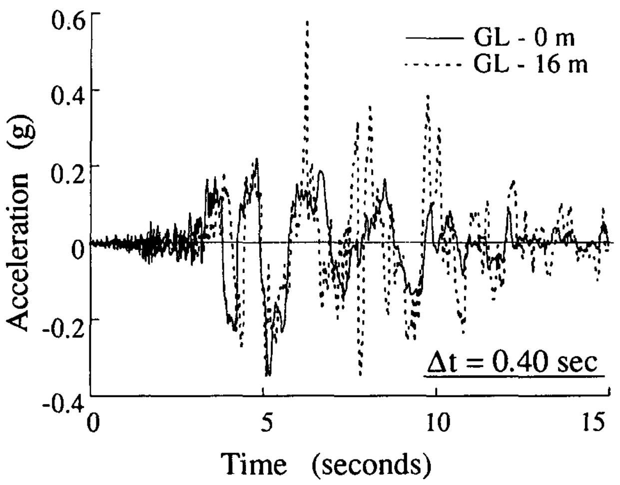

Fig. 4 Comparison between the record at 16 m depth and the surface record shifted for Δt

4.1 Numerical Models

Two series of analyses are conducted by using a couple of numerical models denoted as: 1) Shallow model, and 2) Deep model. Both numerical models are soil-column models composed of four node finite elements.

The shallow model has a height of 28 meters and includes the surface Masado layer and the original sea-bed layer of alluvial clay. The recorded horizontal motion at 32 m depth is used as a base input motion. Four independent analyses are conducted, in the recorded N-S and E-W directions, as well as in the approximate directions of maximum and minimum intensity of the motion, NW-SE and NE-SW, respectively.

The deep model includes the whole profile shown in Fig. 2 and has a height of 83 m. The analyses with the deep model are conducted in the same manner as those of the shallow model, except that the recorded horizontal motion at 83 m depth was used as a base input motion. The top 28 m of the model are identical with the shallow model.

4.2 Material Modeling

In order to facilitate the material modeling only Masado and the alluvial gravelly sand are modeled with the S-D Model, while the clays and the diluvial gravelly sand are modeled as linear materials with degraded stiffness according to the data listed in Table 1. Dilatancy parameters of

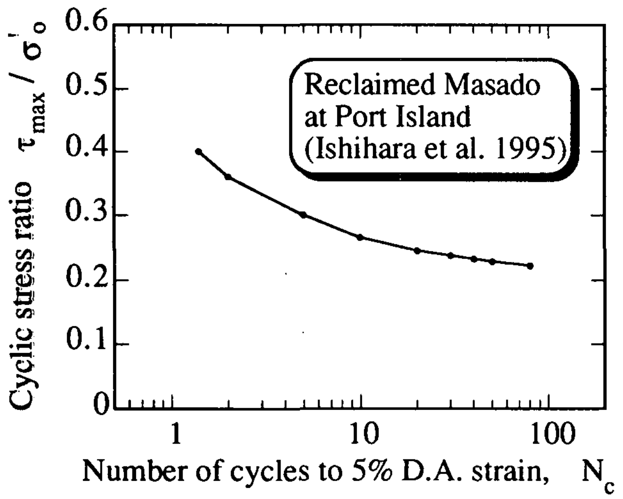

Fig. 5 Liquefaction resistance of reclaimed Masado at Port Island (Ishihara et al., 1995)

Masado were determined by simulating the liquefaction resistance of Masado proposed by Ishihara et al. (1995), shown in Fig. 5. This curve is derived by compiling results from two sets of cyclic triaxial compression tests on undisturbed samples of Masado from Port Island. Due to insufficient laboratory data for Masado, the stress-strain and the state index parameters of the S-D Model were approximated with those of Toyoura sand with relative density of D_r = 50 %. For the same reason, S-D Model parameters for the gravelly sand were identical to those of Masado, except for the dilatancy parameter which was varied in a parametric manner, as explained in the subsequent section.

5. RESULTS AND DISCUSSION

5.1 Shallow Model Analysis

Results of the shallow model analysis will be firstly used to assess the accuracy of the effective stress analysis. For this reason, Figs. 6 and 7 comparatively show the computed and the recorded acceleration time histories at the ground surface and at 16 m depth, for N-S and E-W directions respectively. It can be seen that the computed motion at the ground surface simulates very well all the important features of the corresponding recorded motion such as the peak accelerations, number of intensive cycles and diminishing of the motion due to liquefaction. The similarity between the computed and the recorded motions at 16 m depth is especially remarkable.

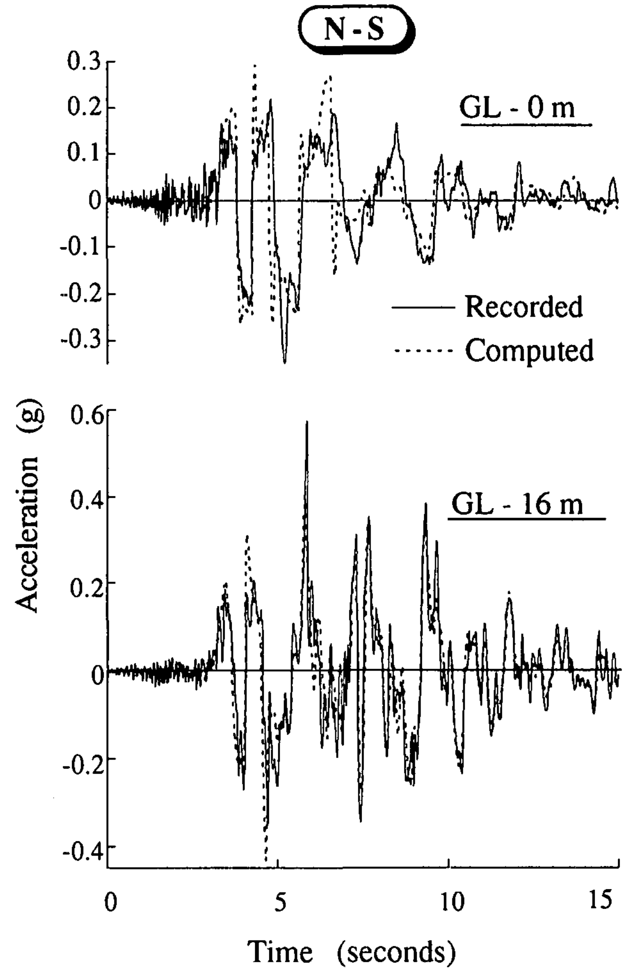

Fig. 6 Computed and recorded N-S acceleration time histories at the ground surface and 16 m depth

The trajectory of the surface motion in the horizontal plane shown in Fig. 8 displays that directionality of the motion is very pronounced. Even though some deviation is evident in Fig. 8, for the purpose of the current effective stress analysis we will assume that NW-SE and NE-SW are the directions of the maximum and the minimum shaking intensity, , respectively. It is important, therefore, to examine the responses for the two extreme intensities of the motion as indicated above.

Figs. 9 and 10 show comparison of the computed and recorded motions at the ground surface and at 16 m depth, for the directions of maximum (NW-SE) and minimum (NE-SW) shaking intensities, respectively. Apparently, the computed and recorded motions for NW-SE direction are very similar. However, the computed ground surface motion for the minimum

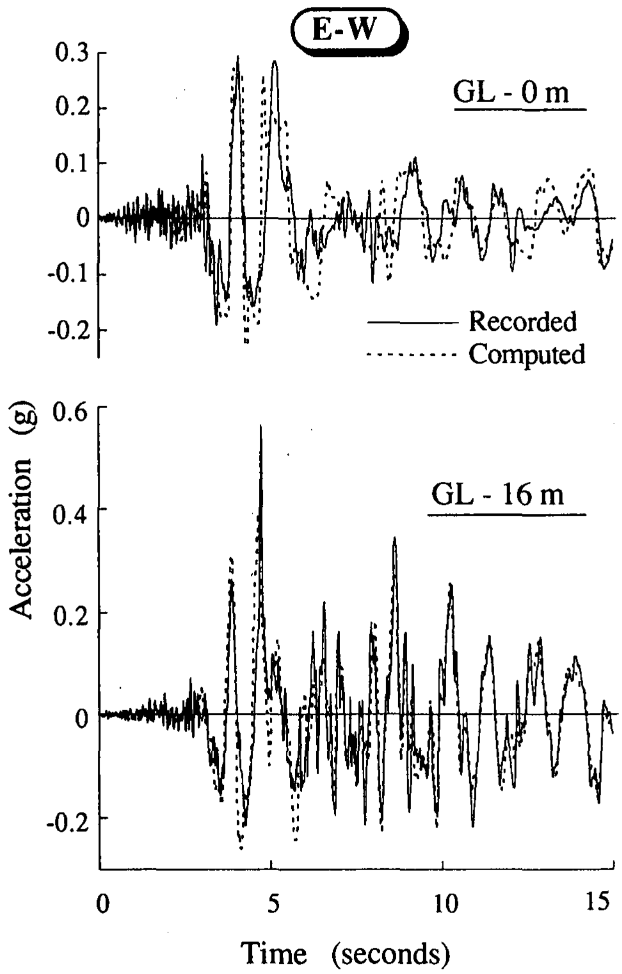

Fig. 7 Computed and recorded E-W acceleration time histories at the ground surface and 16 m depth

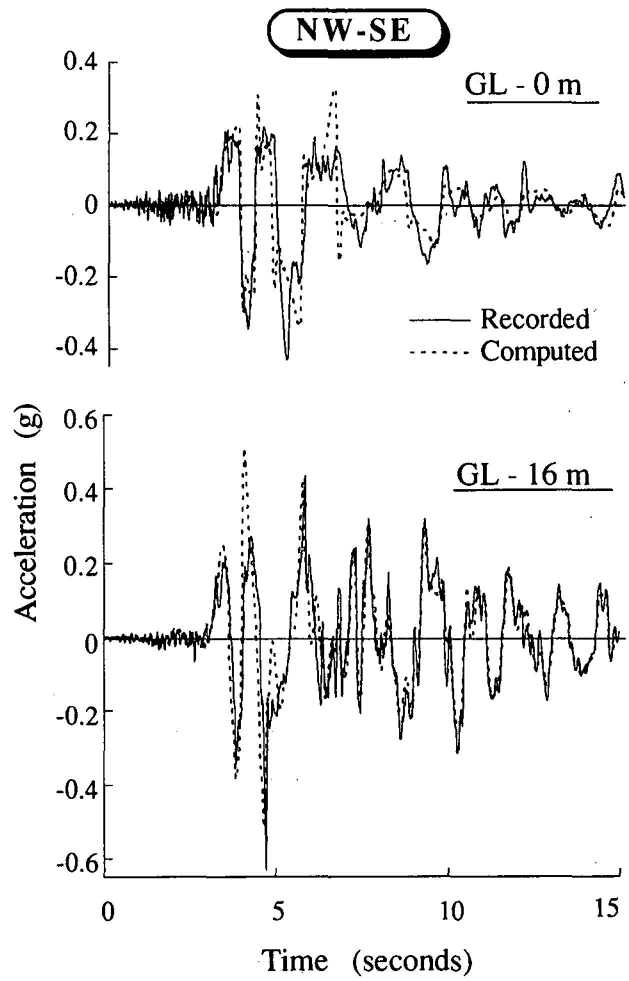

Fig. 9 Computed and recorded NW-SE acceleration time histories at the ground surface and 16 m depth

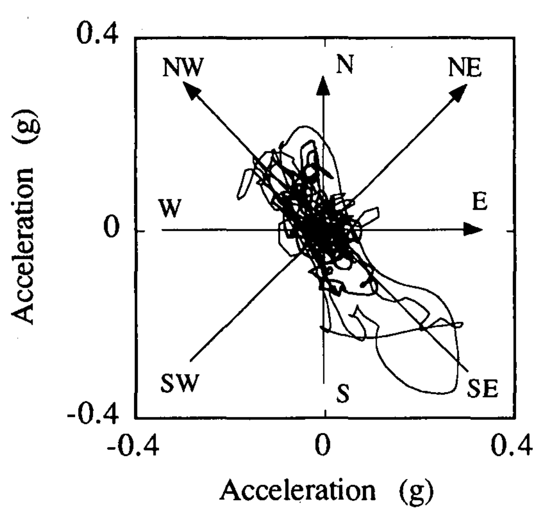

Fig 8 Locus of the ground surface motion

shaking intensity direction, NE-SW, is completely different from the observed one. The reason for such result is clarified in the following.

The only difference between the above independent 1-D analyses has been the base input motion which was changed according to the analyzed direction. The significant differences in the intensity of the motion resulted in very different pore pressure responses, as illustrated in Figs. 11 and 12a where computed excess pore pressure time histories at 10.5 m depth and distribution of their maximum values along the depth of Masado layer, respectively, are shown. The pore pressure and shear strain responses of the N-S and E-W analyses were very similar, and therefore, only the results of the N-S analysis are used herein. A detailed examination of the pore pressure responses reveals that, in the NW-SE analysis, the pore pressure build-up was slightly faster, and the dilation was stronger than those of the N-S analysis. Certain difference in the extent of the excess pore pressures in the shallow part of Masado is also evident. Effecti-

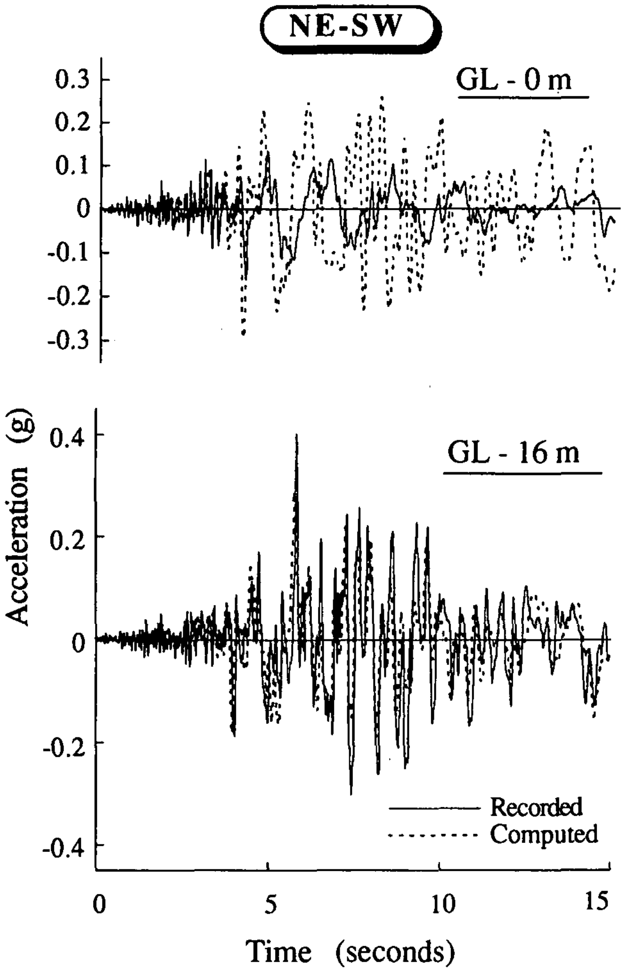

Fig. 10 Computed and recorded NE-SW acceleration time histories at the ground surface and 16 m depth

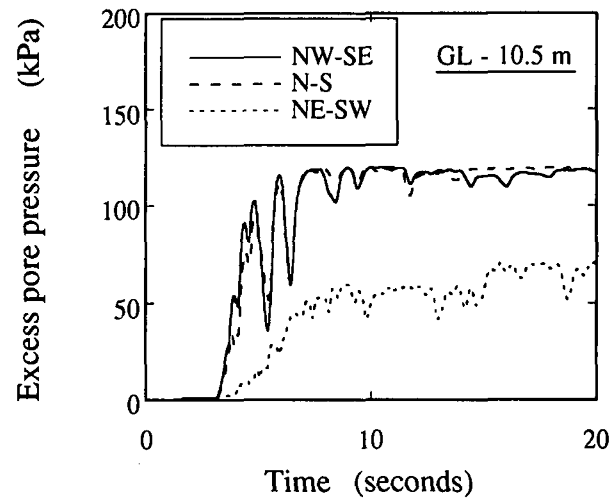

Fig. 11 Computed excess pore pressure time histories at 10.5 m depth

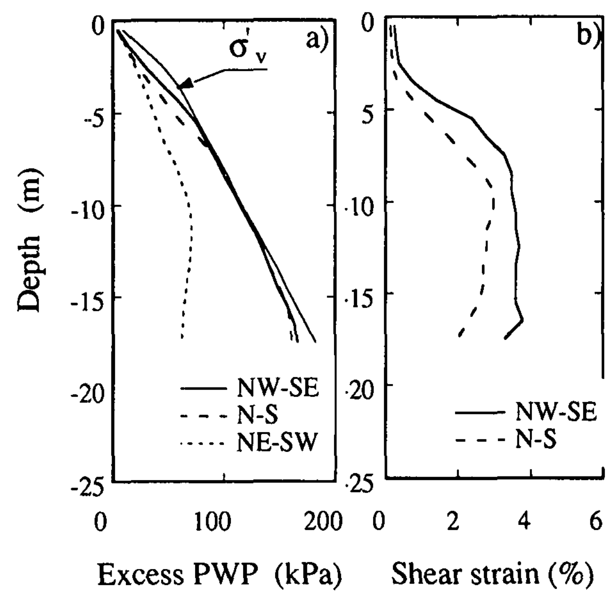

Fig. 12 Distribution of maximum excess pore pressures and shear strains in Masado

vely, however, the pore pressure responses of these two analyses could be considered as similar. On the other hand, the pore pressure response of the minimum intensity analysis (NE-SW) is very different in both development and extent.

The results of the N-S and NW-SE analyses indicate that, approximately from 5 m to 15 m depth of the Masado layer, the excess pore pressures verge on the initial effective vertical stresses, and this response is associated with maximum shear strains of 3-4 %, as shown in Fig. 12b. It is important to recognize that the maximum intensity component of the motion NW-SE is solely sufficient to induce such a response. Similarity among the pore pressure responses of the N-S, E-W and NW-SE analyses, and the fact that the computed motion by these analyses is very close to the observed one, indicate that the computed pore pressure response in these analyses is close to the actual one. Due to much smaller intensity of shaking in the NE-SW direction, the corresponding analysis failed to achieve the actual excess pore pressures, and this resulted in different acceleration response from the observed one.

An important bearing from these results for a liquefaction analysis that uses only one horizontal input component, such as 1-D soil-column analysis or 2-D plane strain analysis, has to be recognized. Namely, the excess pore pressure, as a scalar quantity, has to be associated with the characteristics of the response in the direction of maximum shaking intensity.

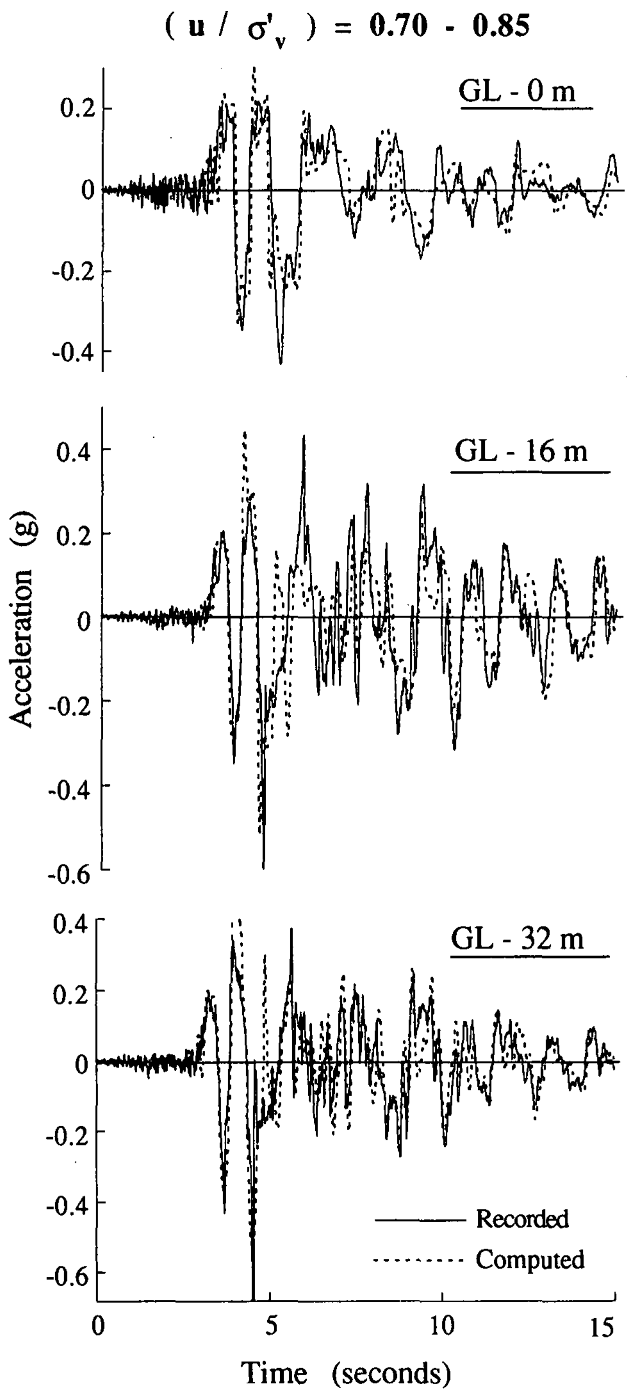

Fig 13 Computed vs. recorded acceleration time histories (Deep model: Analysis A)

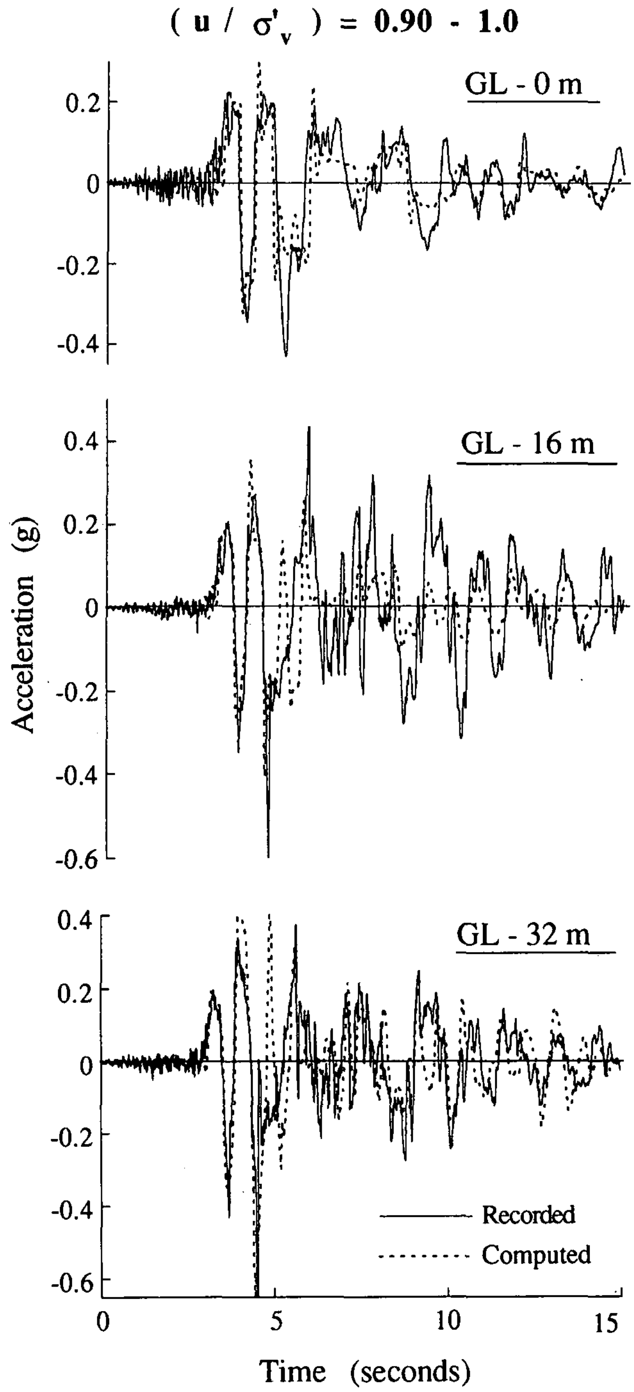

Fig. 14 Computed vs. recorded acceleration time histories (Deep model: Analysis B)

The inertial characteristics of the response, on the other hand, has to be associated with the characteristics of the motion in the considered direction. While the former is illustrated through the elaborated inappropriateness of the NE-SW analysis, the later is displayed in Fig. 12b where despite the similar pore pressure responses in the N-S and NW-SE analyses, the shear strain response is larger for the NW-SE analysis due to larger shaking intensity in this direction. The combination of the inertial and pore pressure response characteristics as above is important for proper liquefaction analysis and accurate evaluation of the seismic responses of ground and soil-structure systems.

5.2 Deep Model Analysis

One of the important issues of the ground response at Port Island is the pore pressure response of the gravelly sand laying between

28m and 37m depth. To investigate this response the deep model analysis was employed. Since laboratory data for the gravelly sand were not available and, on the other hand, the numerical modeling of Masado was established through the shallow model analysis, a parametric study was conducted including a series of independent analyses in the NW-SE direction. In each analysis the dilatancy property of the gravelly sand was different and chosen such to induce different excess pore pressure level in the gravely sand layer. In such manner, effects of the pore pressure response of the deep gravelly sand on the overall acceleration response were examined.

Figs. 13 and 14 comparatively show the recorded and the computed acceleration time histories at the ground surface, 16 m and 32 m depth, in two typical analyses with the deep model. As indicated in these figures, the excess pore pressure in the gravelly sand reached about 60-85 % (Analysis A) and 90-100% (Analysis B) of the initial effective vertical stress, in the analyses shown in Fig. 13 and Fig. 14, respectively. Results of the Analysis A indicate that the recorded motions at all depths can be closely simulated when the pore pressure in the gravelly sand layer is in the range between 60-85 % of the initial effective vertical stress. The analysis B, which could be considered as a case in which complete liquefaction developed in the deep gravelly sand, failed to emulate the recorded motion at 16 m depth. The computed motion at this depth shows clear signs of the liquefaction in the deep gravelly sand such as diminishing of the motion after two to three cycles. It is interesting to note that in both analysis the computed motion at the ground surface was very similar with the recorded surface motion. Based on the results from the deep model analysis it can be concluded that the excess pore pressure in the deep gravelly sand was less than 90 % of the initial effective vertical stress, and therefore, this layer did not undergo a complete liquefaction. The analysis indicates, however, that high pore pressures in the range between 60-85 % of the initial effective vertical stress were likely to be induced in this layer. This response was associated with maximum shear strains of about 1 %. Having in mind the complex stratification of the gravelly sand with some silt layers, and the importance of the stress-strain-dilatancy relation for proper evaluation of soil response, further study is needed to support and strengthen the above findings.

6. CONCLUSIONS

A series of effective stress analyses using shallow and deep models are conducted in order to investigate the ground response at Port Island induced by the Hyogoken-Nanbu Earthquake.

Both recorded motions and results from the analyses indicate that cyclic softening due to liquefaction in Masado layer occurred after one and a half to two cycles of intensive shaking. The analyses further reveal that most heavily affected by the liquefaction was the part from approximately 5 m to 15 m depth. In this part of the Masado layer the excess pore pressure reached the initial effective vertical stress, and maximum shear strain of 3-4 % developed.

There is no indication in the recorded motions that similar liquefaction took place in the alluvial gravelly sand. In fact, the motion at 16 m depth preserved the shape of the motions at the two lower levels. The deep model analyses disclose that high pore pressures in the range between 60-85 % of the initial overburden pressure associated with maximum shear strains of about 1 % were likely to develop in this layer.

The shallow model analyses in directions with different shaking intensity demonstrate the large influence of the excess pore pressures on the overall ground response. The results of these analysis emphasize the need to consider the maximum shaking intensity component of the motion for evaluation of the pore pressure response.

REFERENCES

Ansary, M. A., Yamazaki, F., Katayama, T. and Towhata, I., 1995. Analysis of ground motions at a liquefied site during the 1995 Great Hanshin Earthquake. 23rd Earth. Engrg. Workshop,JSCE.

Cubrinovski, M. 1993. A constitutive model for sandy soils based on a stress-dependent density parameter. D. Eng. Thesis, University of Tokyo.

Ishihara, K., Yasuda, S. and Nagase, H. 1995. Soil characteristics and ground damage. *Soils and Foundations* : to be published.

Toki., K. 1995. Committee of earthquake observation and research in Kansai area.