ANALYSIS OF TWO CASE HISTORIES ON LIQUEFACTION OF RECLAIMED DEPOSITS

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SUMMARY

Two well-documented case histories on liquefaction of reclaimed land from the 1995 Kobe earthquake are discussed in this paper. The two sites (VA-site and PH-site) are in proximity to each other and have practically identical stratification with an 18 m thick fill layer overlying an alluvial clay layer. At the VA-site, no ground improvement measures have been undertaken and the reclaimed layer has a relatively low SPT blow count of 5 to 10 throughout the depth. The fill layer at the PH-site, on the other hand, has been improved by means of the rod compaction method resulting in increase in the SPT resistance to approximately 20 to 40 blow counts. To investigate the differences in the ground responses between the densified and undensified fills during the Kobe earthquake, analyses of the sites were conducted by using an effective stress method. It was found that both the extent and characteristics of the liquefaction were significantly different at the two sites. The undensified fill layer completely liquefied below the water table and developed maximum shear strains of about 4 % and settlement of 25 cm. On the other hand, only the deep part of the densified Masado layer liquefied resulting in maximum shear strains of 2 % and settlement of approximately 8 cm.

INTRODUCTION

Reclaimed lands are known to be highly susceptible and vulnerable to liquefaction. A recent reminder of the vulnerability of reclaimed soils to liquefaction was the 1995 Kobe earthquake (Hyogoken-Nambu earthquake) during which widespread liquefaction occurred in reclaimed lands in the coastal area of the city of Kobe. The exceptionally extensive and massive liquefaction of thick fill deposits resulted in large ground deformations, failure of foundations and slumping of revetment lines. One important observation from the Kobe earthquake is the fact that fewer signs of liquefaction and lesser ground deformation were found in reclaimed deposits that have been treated by soil improvement methods. The objective of this paper is to investigate the performance of the reclaimed fills during the Kobe earthquake and to highlight the difference in the ground responses between densified and undensified reclaimed deposits. This study is based on two well-documented case histories from the Kobe earthquake and combines comprehensive in-situ investigations, laboratory testing and observations from the earthquake with results of effective stress analyses.

2. SITE CHARACTERISTICS

The two investigated sites are located in the northwest part of the Kobe Port Island (Fig. 1a), a man-made island which is in proximity to the causative fault of the 1995 Kobe earthquake and is only few kilometers south of the zone of the city of Kobe that has been the most heavily damaged by the quake. The land of Port Island, with an area of 436 ha, was reclaimed in the period between 1966 and 1980 by using a weathered gravelly soil (Masado) as a fill material. The land filling resulted in a 15-20 m thick fill deposit.

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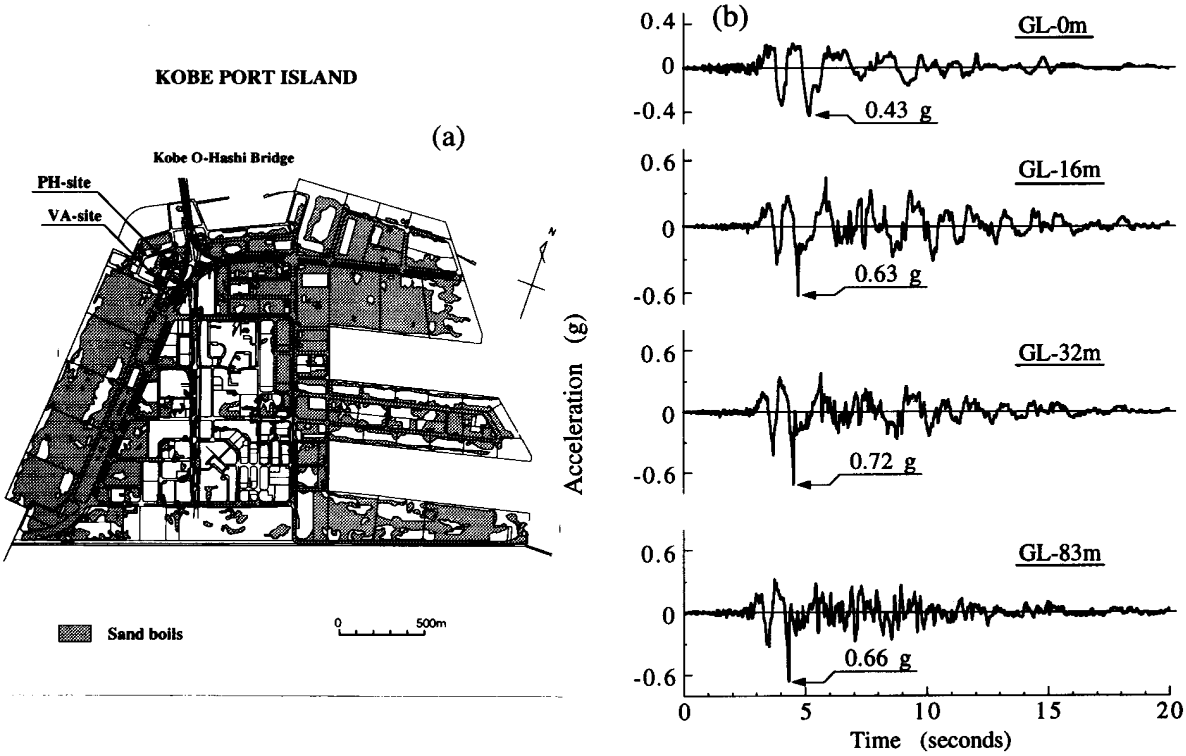

Figure 1. Location of the investigated sites and observed accelerations during the 1995 Kobe earthquake: (a) Kobe Port Island; (b) recorded ground accelerations at the vertical array site (NW-SE direction)

2.1 Seismic Vertical Array Site (VA-site)

At the VA-site, a vertical seismic array was installed in 1991. The array consists of four sets of strong motion accelerometers positioned at the ground surface and at depths of 16, 32 and 83 m. Figure 1b shows accelerations of the main shock of the quake as recorded by the array accelerometers. The ground motion at this site exhibited very pronounced directionality, with the maximum shaking intensity being oriented approximately in the northwest-southeast direction.

Soil profile, shear wave velocity and SPT measurements of the vertical array site are shown in Fig. 2. The fill deposit of Masado is 18 m thick, and the ground water level is estimated at 2.8 to 3.5 m depth. The reclaimed fills overlie a 10 m thick layer of alluvial clay which is identified as the seabed deposit before the reclamation. At this site, no ground improvement measures have been undertaken and the Masado layer has a relatively low SPT blow count of 5 to 10 throughout the depth.

The violent shaking by the main shock of the quake caused extensive and massive liquefaction of undensified fill deposits of Port Island resulting in an average settlement of about 30-40 cm and a 15-20 cm thick layer of sand and mud littered on the ground surface. Based on the acceleration records and numerous analyses of the vertical array site (Cubrinovski et al., 1996), it has been found that the maximum relative horizontal displacement within the Masado layer reached about 40 to 50 cm. The observed settlement at the VA-site was about 30 cm. Clear signs of liquefaction at this site are also apparent in the recorded accelerations shown in Fig. 1b as indicated by the decrease in the amplitudes, loss of high frequency response and elongation of the predominant period of the motion at the ground surface.

2.2 Packing Cooperative Site (PH-site)

Unlike the liquefaction of undensified Masado deposits, fewer signs of liquefaction and only scattered sand boils were observed in the areas of improved deposits of Port Island. This is clearly illustrated in Fig. 1a, where most of the central part of Port Island which is free of sand boils in fact coincides with the areas where soil improvement had been executed (Yasuda et al., 1996). The settlement of the densified fills has been also found to be much smaller than that of the untreated deposits. In order to investigate in more details the effectiveness of the

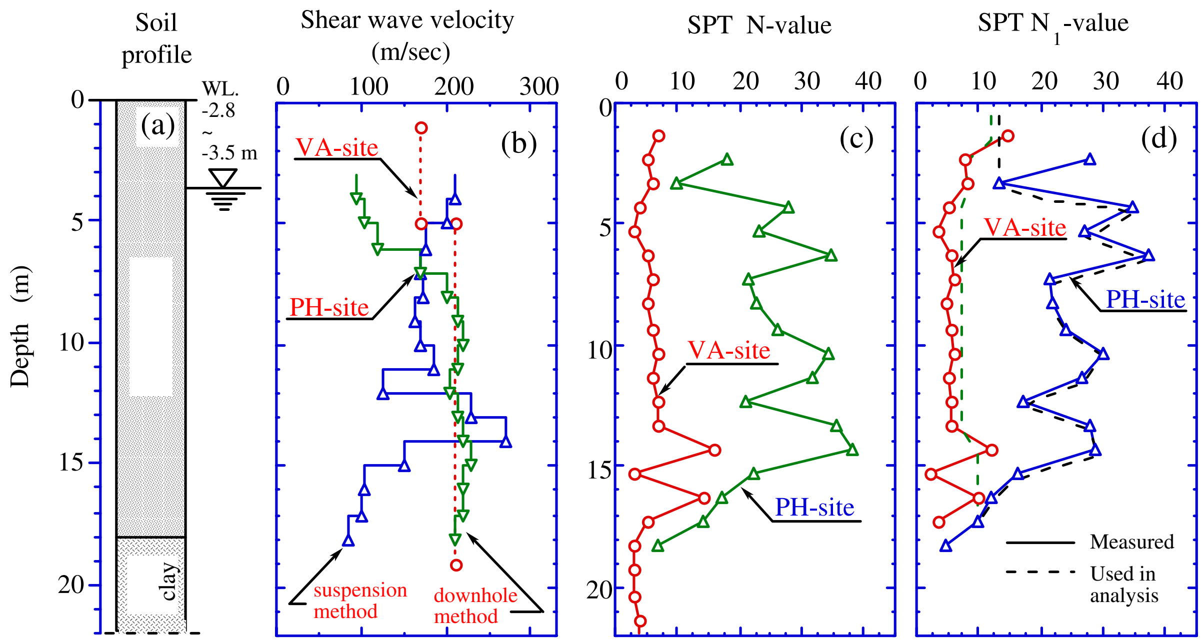

Figure 2. Soil profiles of the investigated sites: (a) soil profile; (b) shear wave velocity; (c) SPT *N* -values; (d) SPT N_1-values

ground improvement, a comprehensive investigation of a selected site has been undertaken by the Geotechnical Research Collaboration Committee on the Hanshin-Awaji Earthquake (Ishihara et al., 1998b). The investigated site, denoted as the Packing Cooperative Site (PH-site), is located also in the northwest part of Kobe Port Island and is about 130 m north of the vertical array site (Fig. 1a). Except for the density of the Masado layer, the soil stratification and fill materials at the PH-site are practically identical to those of the VA-site.

The PH-site, however, has been improved by means of the rod compaction method by employing a regular triangle configuration with a pitch of 2.4 m (Ishihara et al., 1998b). As a result of the ground improvement, the SPT resistance (N-value) significantly increased and reached values on the order of 20 to 40 blow counts. The penetration resistance of the PH-site shown in Fig. 2 is the average of three independent measurements conducted within a few meters distance from each other. Note that the compaction at the PH-site has been executed to a depth of 15 m, and therefore below this depth, the SPT resistance sharply reduces to about 10 blow counts and approaches the values measured at the VA-site. As shown in Fig. 2d, the SPT resistance normalized to an overburden pressure of 98 kPa (N_1 -value) is very different for the two sites and takes values of about 5-10 and 20-30 for the VA-site and PH-site respectively. The shear wave velocity of the Masado layer was found to be predominantly in the range between 150 and 200 m/sec. Sand boils were not observed at the improved site, and the settlement was found to be around 8 cm (Ishihara et al., 1998b).

3. MATERIAL PROPERTIES

The fill material used for the reclamation (Masado) is a well-graded soil containing a fairly large portion of gravel. Gradation curves of Masado samples recovered from the PH-site and a site south of the vertical array (Ishihara et al., 1998a) are shown in Fig. 3a where it may be seen that the gravel fraction of the soil ranges between 37.1 and 63.2 %, while the fines content is from 4.5 to 12.6 % by weight.

After the Kobe earthquake, a number of comprehensive laboratory studies have been conducted to investigate the undrained behaviour and liquefaction characteristics of Masado soils. Results from a series of cyclic undrained tests on undisturbed samples recovered by the ground freezing technique from Port Island are shown in a summarized form in Fig. 3b. Here, the SPT resistance corresponding to each sample or group of samples is also indicated. The large variation of the SPT resistance in the range between 7 and 34 blow counts is due to the fact that the data shown in Fig. 3b contain results of tests on samples recovered from both undensified and densified deposits. Note that the data also include samples recovered from the PH-site as well as a site located in vicinity of the VA-site. It is evident in Fig. 3b that the cyclic strength of Masado soils increases with the SPT resistance.

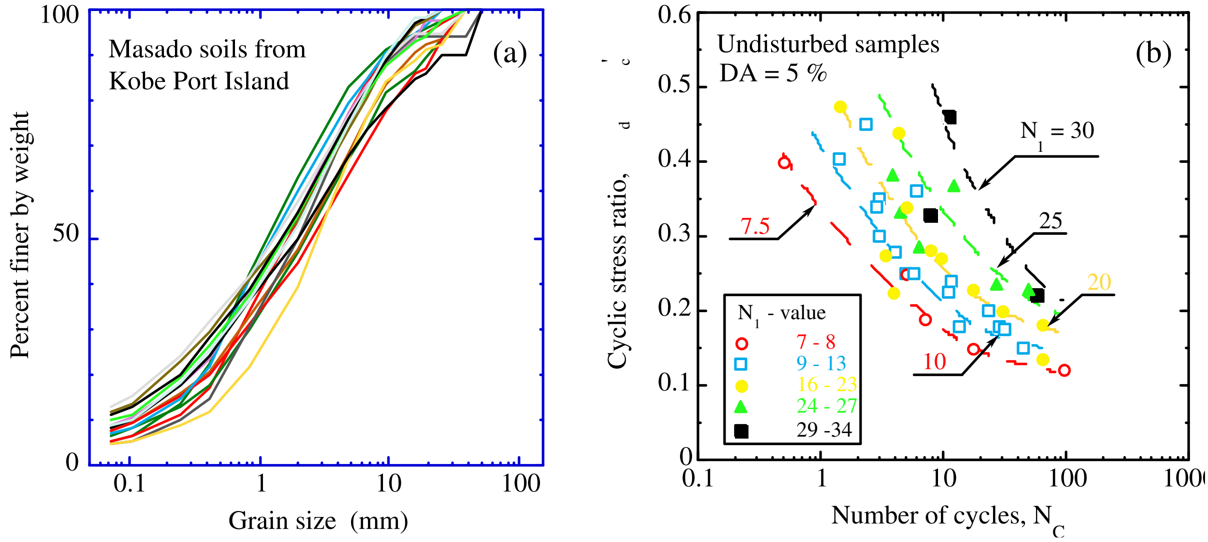

Figure 3. Grain-size distribution and liquefaction resistance of Masado soils: (a) gradation curves; (b) cyclic strength of high-quality undisturbed samples

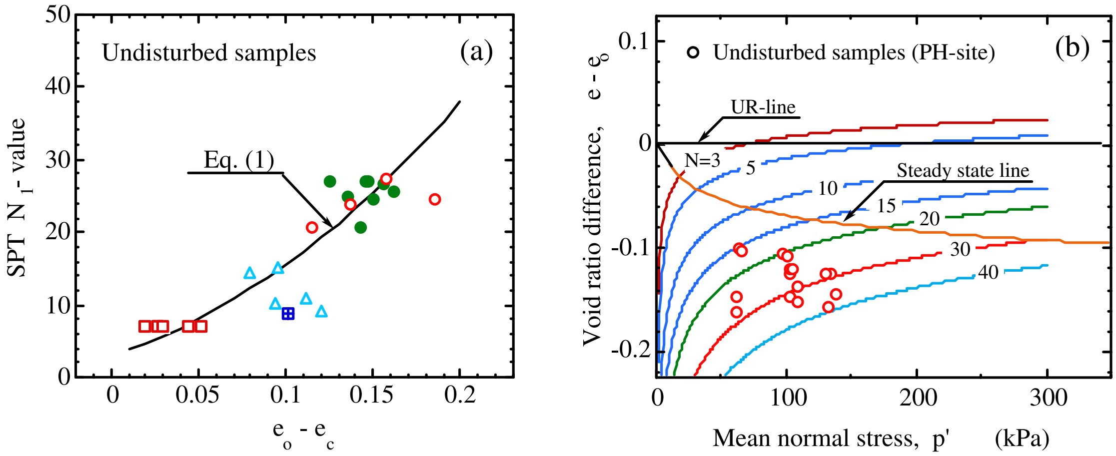

Figure 4. State-concept interpretation of field deposits of Masado: (a) correlation between N_1-value and void ratio of undisturbed samples; (b) in-situ states of the soil at the PH-site relative to the steady state line

The large gravel content and relatively wide variation in the grain-size distribution of Masado soils cause difficulties in evaluating the relative density of these soils and complicate the conventional characterization of soils based on D_r. Within the framework of a comprehensive study on the undrained behaviour and steady state characteristics of Masado soils, Ishihara et al. (1998a) suggested to employ the steady state concept as an alternative way for characterizing Masado soils. In essence, this method uses the state of the soil relative to the steady state line to estimate the behaviour of the soil upon shear load application. In the aforementioned study, results from a series of monotonic undrained tests on high-quality undisturbed samples have been used to establish the relationship between N_1 and $(e_o - e_c)$ shown in Fig. 4a. Here, e_o is the void ratio of the steady state line at a mean normal stress of 0 kPa ($p' = 0$ kPa) while e_c is the void ratio of undisturbed samples. This relationship between the void ratio and penetration resistance of Masado soils can be expressed as:

$$e_c = e_o - \frac{\sqrt{N_1} - 1.73}{22.1} \qquad (1)$$

By using Eq. (1) it is possible to plot the diagram shown in Fig. 4b and to display the in-situ state of the samples superimposed with N-contour lines and the steady state line of Masado soils. The position of the undisturbed samples relative to the steady state line shown in Fig. 4b indicates that the fill layer of the PH-site would have dilative behaviour upon monotonic shearing.

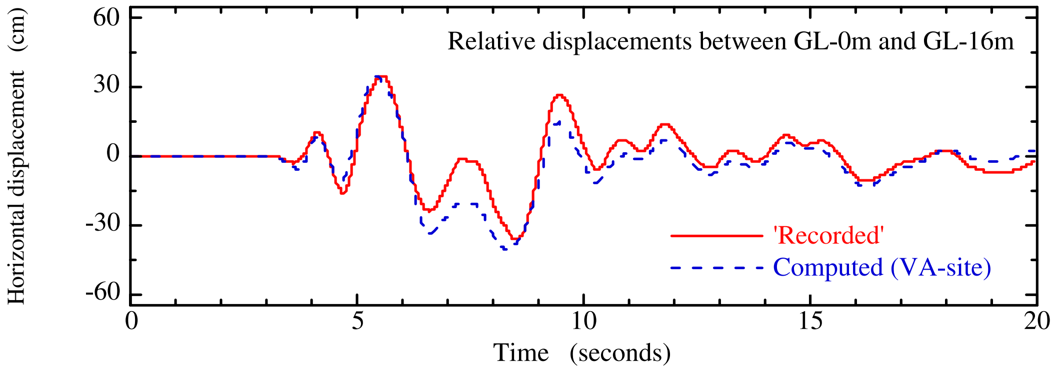

Figure 5. Computed and 'recorded' horizontal displacements of the Masado layer at VA-site

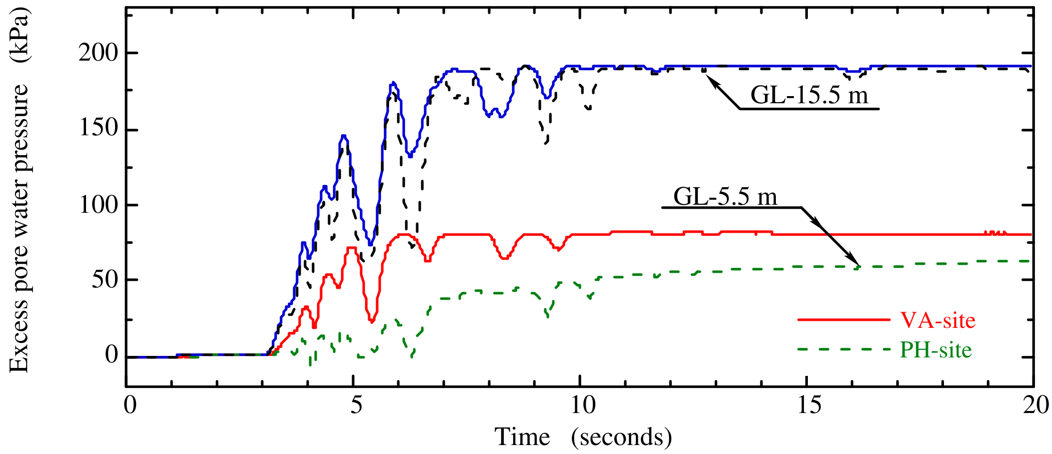

Figure 6. Computed excess pore water pressures at two depths of the fill layer at VA-site and PH-site

4. METHOD OF ANALYSIS

A fully coupled effective stress method of analysis was used to analyze the two sites. In the analyses, the soil-column model consisted of the fill layer and underlying alluvial clay layer, and the recorded motion at 32 m depth in the NW-SE direction was applied as a base input motion. An elastoplastic deformation law for sands which is based on the state concept interpretation of sand behaviour was employed in the analysis (Cubrinovski and Ishihara, 1998). The model parameters for Masado soils, listed in Table 1, were determined as follows: (1) the elastic parameter A was evaluated based on the measured shear wave velocity of the Masado layer while the values of n and ν were assumed; (2) the stress-strain parameters were determined from normalized stress-strain curves measured in drained p'-constant tests; (3) the reference lines (Fig. 4b) were derived from a series of monotonic undrained tests (Ishihara et al., 1998a); (4) finally, the dilatancy parameters were determined by simulating the liquefaction resistance of the undisturbed samples shown in Fig. 3b. A unique feature of this constitutive model is that by using the values of the material parameters listed in Table 1, the behaviour of Masado soils can be modeled for any density of the soil and applied confining stress. Thus, in the analyses of the VA-site and PH-site, the N_1-values were firstly approximated as indicated in Fig. 2d, and then the void ratios throughout the depths of the two profiles were calculated according to Eq. (1). Using these void ratios and the material parameters listed in Table 1, the analyses of the VA-site and PH-site were carried out. Thus, the void ratio was the only different parameter in the analyses of the two sites. The alluvial clay layer was modeled as a nonlinear elastoplastic material with a stress-strain curve defined by a conventional G-γ relationship.

Table 1. Material parameters of Masado soils

Elastic	$A = 199$	$n = 0.80$	$\nu = 0.10$
Stress-strain	$(\tau / p')_{max}$ $a_1 = 0.745$ $b_1 = 0.10$	$G_{N,max}$ $a_2 = 332$ $b_2 = 60$	$G_{N,min}$ $a_3 = 180$ $b_3 = 10$
Reference lines	<u>UR-line</u> $e_o = 0.430$	<u>Steady state line</u> $e_s = 0.464 - 0.051 \log p'$	
Dilatancy	$\mu_o = 0.18$	$M = 0.75$	$S_c = 0.012$

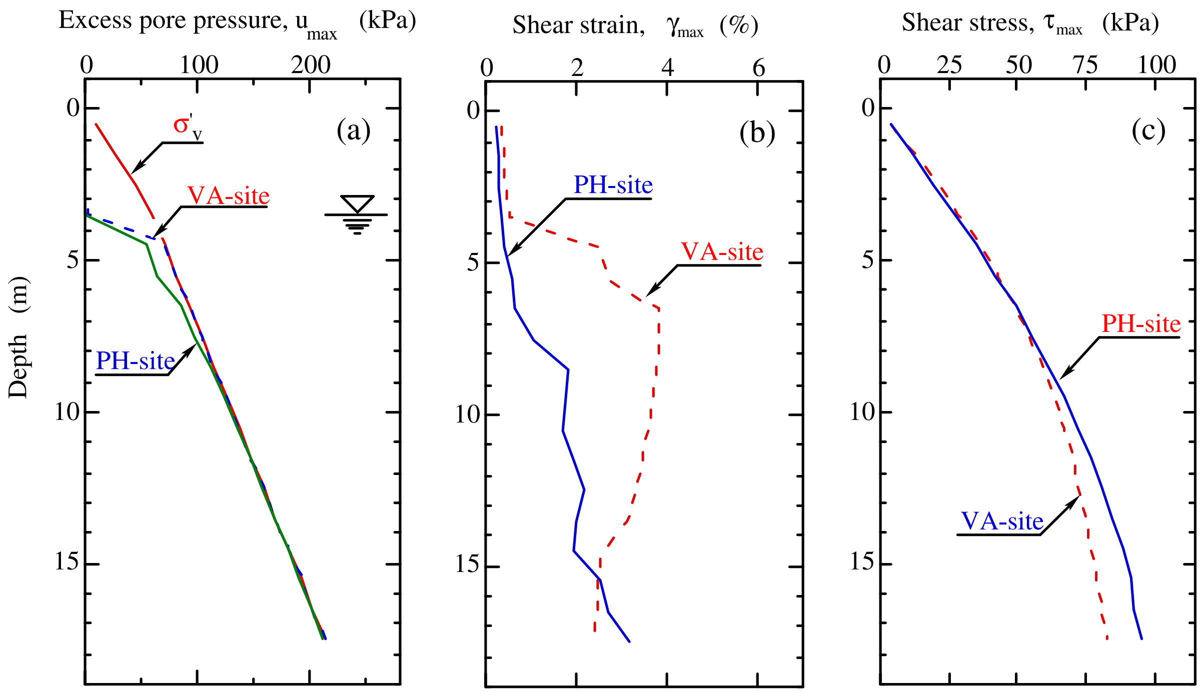

Figure 7. Computed maximum ground responses at the VA-site and PH-site: (a) excess pore pressures; (b) shear strains; (c) shear stresses

5. COMPUTED GROUND RESPONSE

A series of effective stress analyses of the vertical array site have been previously presented by the authors where it has been shown that the observed ground accelerations at this site can be successfully simulated by the employed numerical procedure (Cubrinovski et al., 1996). To illustrate this aspect of the analysis, Figure 5 comparatively shows the computed and 'recorded' relative displacements of the fill layer which is the difference in displacements between the ground surface and 16 m depth. Note that the 'recorded' displacements were actually computed through a double integration of the recorded accelerations shown in Fig. 1b. In what follows, the computed ground responses in the analyses of the VA-site and PH-site are comparatively presented.

5.1 Excess Pore Pressures

The pore pressure build-up computed at two different depths of the fill layer is shown in Fig. 6 whereas distribution of the maximum excess pore pressures along the depth of the deposit is shown in Fig. 7a. These results reveal that even though the fill layer liquefied at both sites, the extent of the liquefaction was different. Namely, at the VA-site, the Masado layer completely liquefied below the ground water level, whereas in the case of the PH-site, the excess pore pressures reached the effective overburden stress level or 100 % only at depths greater than 8 m. Thus, at the PH-site, the layer between 3.5 and 8 m developed high excess pore pressures, but did not liquefy during the shaking. Scrutiny of the excess pore pressures shown in Fig. 6 further reveals that at 15.5 m depth the pore pressure development was very similar for the two investigated sites which is due to the similarities in the soil properties at this depth. On the other hand, at 5.5 m depth, the pore pressure build-up was much slower at the PH-site.

5.2 Shear Stresses and Strains

Computed maximum shear strains and stresses along the depth of the fill layer are shown in Figs. 7b and 7c respectively. It is apparent that within the liquefied part of the deposits, the maximum shear strains at the VA-site reached about 3.5 to 4.0 % whereas the corresponding shear strains at the PH-site were around 2 %. Thus, even though liquefaction was not completely prevented, the soil improvement reduced the shear strains and limited the deformation of the liquefied layer. At depths shallower than 8 m, the maximum shear strains at the PH-site were much smaller and further reduced due to the limited excess pore pressures in this portion of the Masado layer. Characteristics of the ground response as described above are outlined in the computed stress-strain

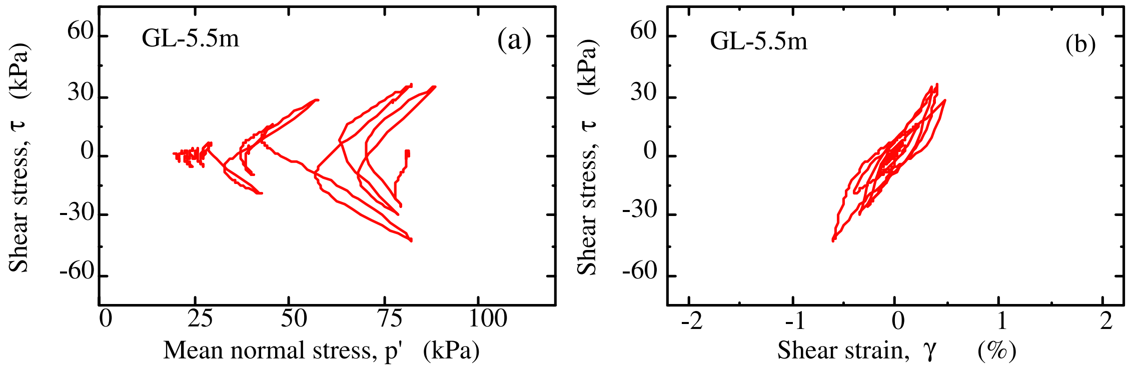

Figure 8. Computed stress-strain response at 5.5m depth of the PH-site: (a) stress path; (b) stress-strain curve

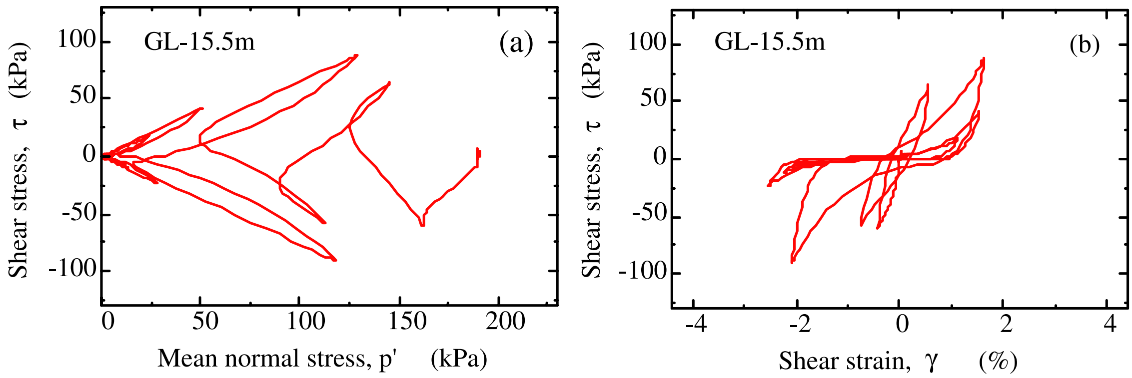

Figure 9. Computed stress-strain response at 15.5m depth of the PH-site: (a) stress path; (b) stress-strain curve

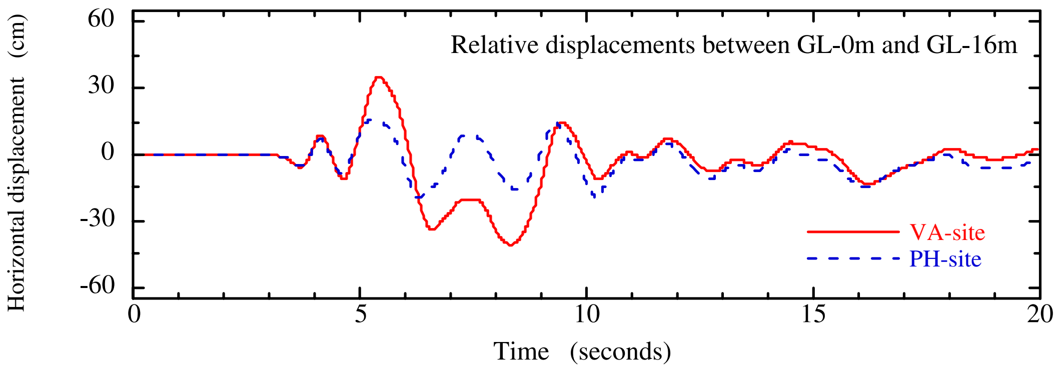

Figure 10. Comparison between the relative ground displacements of the fill layer at VA-site and PH-site

curves and effective stress paths presented in Figs. 8 and 9. One consistent difference between the responses of the VA-site and PH-site was that the behaviour of the improved ground produced more dilative and 'stiffer' response, resulting in higher accelerations at the ground surface and higher shear stresses in the deep part of the fill layer at the PH-site (Fig. 7c).

5.3 Displacements and Settlements

In accordance with the ground deformation characteristics as described above, the relative displacement within the Masado layer was found to be much smaller at the improved site as compared to the undensified deposit. As shown in Fig. 10, the peak horizontal displacements were found to be around 20 cm and 40 cm, for the PH-site and VA-site respectively.

Based on the results of the effective stress analyses, a simple calculation of the settlements was carried out according to the procedure proposed by Ishihara and Yoshimine (1992). This calculation is based on a correlation

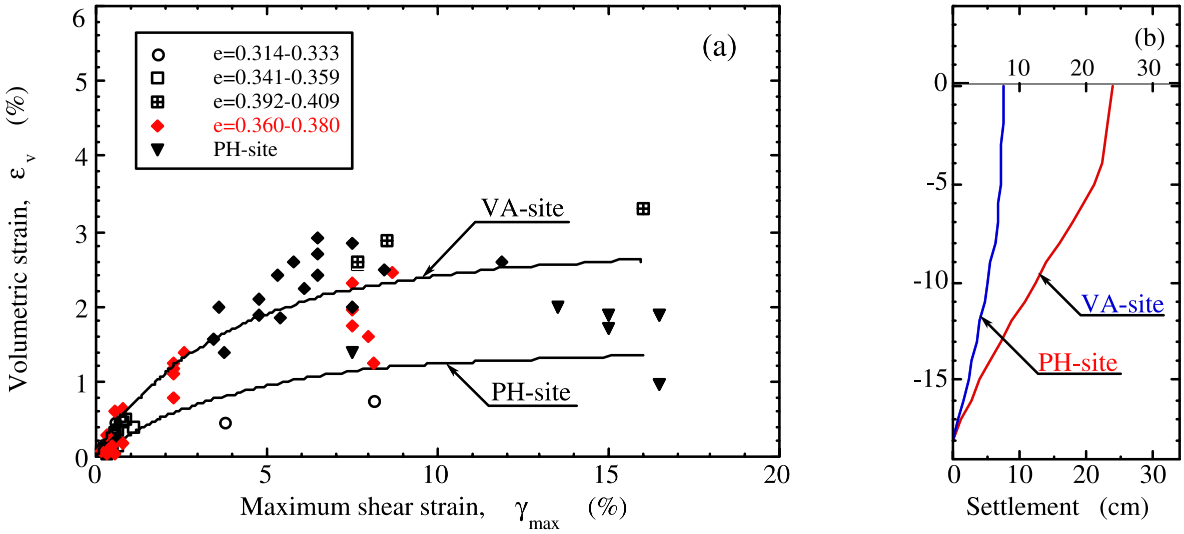

Figure 11. Simplified estimation of settlements: (a) ε_v -γ_{max} relationship; (b) distribution of settlements

between the volumetric strain resulting from dissipation of excess pore pressures following undrained cyclic loading and the maximum shear strain attained in the cyclic loading phase of the test. Such correlation obtained from tests on samples of Masado soils is shown in Fig. 11a, where two representative relationships, for the VA-site and PH-site, are also indicated. By using the maximum shear strains computed in the effective stress analyses (Fig. 7b), the volumetric strains for each computational layer were read off from the chart of Fig. 11a, and subsequently the settlements were calculated by integrating the volume changes throughout the depth. As shown in Fig. 11b, the computed settlements of the ground surface are 7.7 cm and 24.1 cm for the PH-site and VA-site respectively, which are in reasonable agreement with the observed settlements at these sites.

6. CONCLUSIONS

Results of the effective stress analyses reveal that the extent and characteristics of the liquefaction induced by the 1995 Kobe earthquake were remarkably different at the VA-site and PH-site. The undensified fill layer of the VA-site completely liquefied below the water table or 3.5 m depth. At this site, the liquefaction was associated with maximum shear strains of 3.5 to 4.0 % and ground settlement of about 25 cm. The densified fill layer of the PH-site, however, liquefied only in the deep part below 8 m depth resulting in maximum shear strains of about 2 % and settlement of approximately 8 cm. Thus, the increased density of the soil by the ground improvement measures prevented the occurrence of liquefaction in the shallow part of the fill deposit and limited the deformation of the deep part of the fill that liquefied during the earthquake. Results of the analyses were found to be in good agreement with the ground observations secured during and immediately after the earthquake.

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