Liquefaction-Induced Ground Deformation and Damage to Piles in the 1995 Kobe Earthquake

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ABSTRACT: A significant geotechnical feature of the 1995 Kobe earthquake was the widespread and massive liquefaction of reclaimed fills in the port area of Kobe. The liquefaction resulted in cyclic ground displacements of inland fills of 30-40 cm while lateral spreading towards the sea occurred in the waterfront area with a magnitude of 1-4 m at the quay walls. The excessive ground movements caused numerous failures and damage to pile foundations in the waterfront area. This paper summarizes the outcome of detailed field, laboratory and analytical investigations and highlights the key features of the liquefaction during the Kobe earthquake. Particular attention is given to liquefaction-induced ground displacements and to their effects on the performance of pile foundations.

KEYWORDS: Kobe earthquake, liquefaction, lateral spreading, reclaimed soils, pile foundations.

1 INTRODUCTION

The Hyogoken-Nambu earthquake (Kobe earthquake), registering a magnitude of 7.2 (JMA), struck the western part of the main island of Japan, on January 17, 1995. The fault rupture started north of Awaji Island (inset of Fig. 1) and progressed through three fault segments delivering a devastating shock to Kobe, a city of 1.5 million people and the second largest port in Japan. About 5500 fatalities have resulted from the earthquake, 300000 people were left homeless, approximately 70000 wooden houses and 3000 commercial and residential buildings were heavily damaged or collapsed (JGS 1998).

A significant geotechnical feature of the 1995 Kobe earthquake was the widespread liquefaction of reclaimed lands in the bay shore area of Kobe. Reclaimed fills with thicknesses of 10-20 m extensively liquefied littering the surface of man-made islands with ejected water and sand. The liquefaction resulted in excessive ground deformation with settlements as large as 30-40 cm and a similar magnitude of cyclic displacements of the ground. The quay walls moved about 1-4 m towards the sea and lateral spreading occurred in the backfill soils that progresses inland to a distance as far as 200 m from the revetment line. A large number of pile foundations of buildings, storage tanks and bridge piers experienced significant cyclic and permanent lateral ground displacements resulting in numerous failures and severe damage to piles (JGS, 1998).

After the earthquake, detailed investigations have been conducted including ground survey measurements, field inspections of damage to piles, laboratory tests on soil samples and numerical studies of well-documented case histories. This paper presents a brief summary of the outcome from these investigations and highlights the key features of the liquefaction during the Kobe earthquake. Particular emphasis is given to liquefaction-induced ground displacements both due to cyclic loads and lateral spreading and to their effects on the performance of pile foundations. Lessons learned and incentives for revision of liquefaction-evaluation codes and seismic design of piles are also addressed.

2 CHARACTERIZATION OF RECLAIMED SOILS

Most of the man-made islands in the port area of Kobe (Fig. 1) have been reclaimed over the period from 1950 to 1980 using decomposed granite rocks from the Rokko Mountains. The soil dubbed

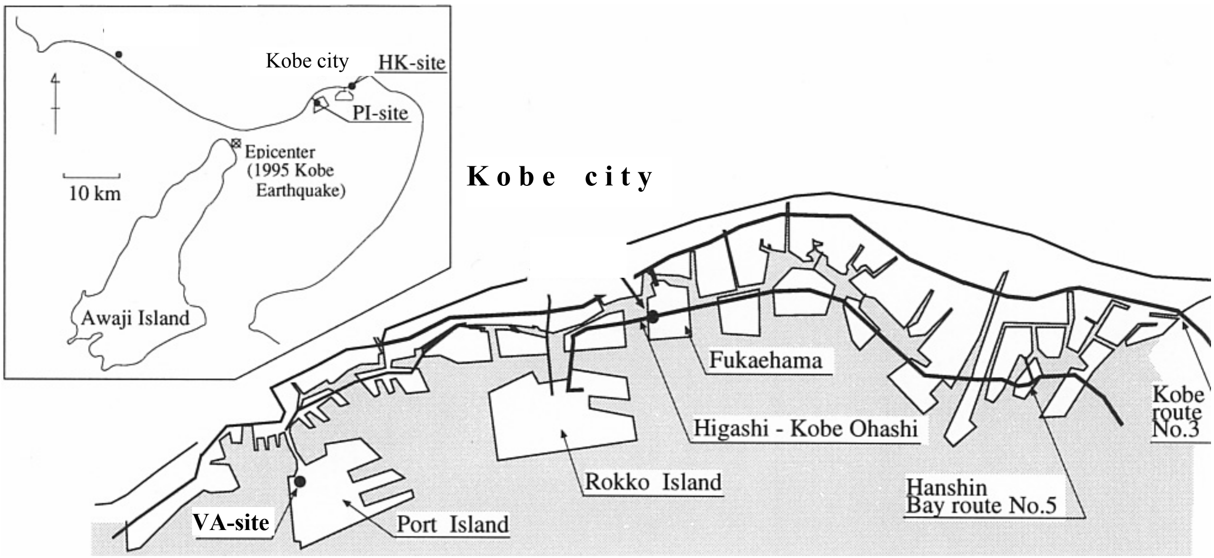

Figure 1. Location of the epicenter (inset) and overview of reclaimed lands in the port area of Kobe

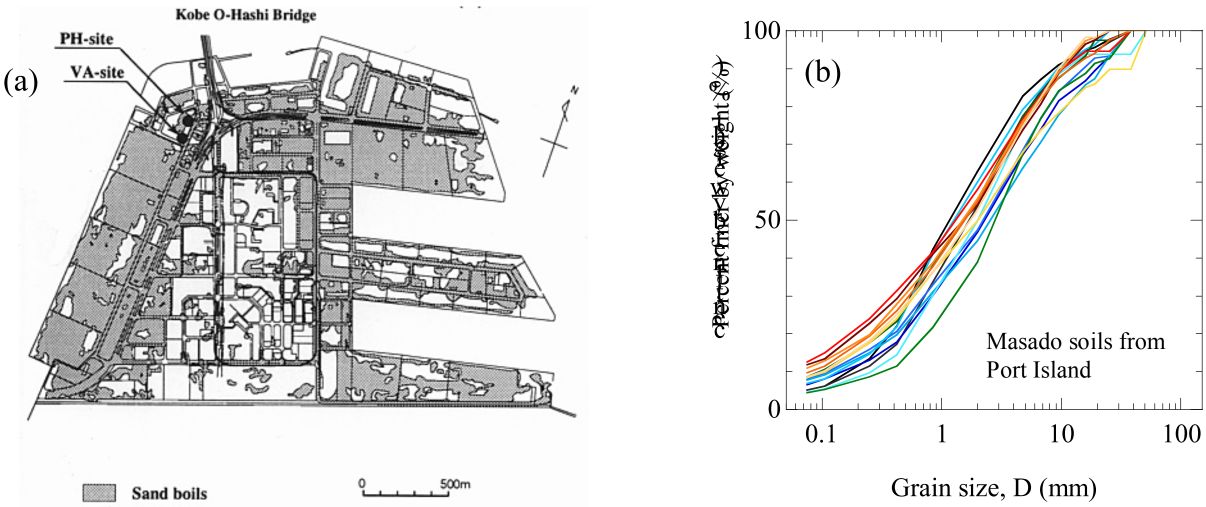

Figure 2. Port Island: (a) map showing areas covered with sand boils; (b) grain-size distribution of fill soils

Masado was transported from the borrow sites by a system of conveyor belts and push-barges and dumped under the water at the site of reclamation. The thickness of the reclaimed fills is about 15-25 m in Port Island and Rokko Island, and less than 15 m in other reclaimed areas. The fill deposits overlie the original seabed layer of Holocene clay which in turn overlies an upper Pleistocene deposit. Figure 2a shows the Port Island, one of the largest man-made islands with an area of about 436 ha.

One distinctive feature of the liquefaction during the Kobe earthquake is associated with the grain composition of the fill soils. Namely, the fill materials are generally well-graded and contain significant portion of gravel particles. Typical grain composition of Masado soils is shown in Fig. 2b where gradation curves of undisturbed samples recovered from fill deposits of Port Island are displayed. The gravel fraction ($D \geq 2$mm) of these soils ranges between 37% and 63% while the fines content ($D < 0.075$mm) is from 4% to 12% by weight. Thus, it was somewhat surprising that massive liquefaction occurred in soils containing a fairly large portion of gravel.

Following the reclamation, in some areas the fill deposits have been improved by preloading or by installing sand drains. Interestingly, these remedial measures have been used as a protection against consolidation settlements of the underlying clay layer rather than as a countermeasure against liquefaction of the fill deposits. In addition, some of the reclaimed fills have been compacted using eifaction of the fill deposits. In addition, some of the reclaimed fills have been compacted using either sand compaction piles or rod (vibro) compaction. Whichever the case is, the ground improvement resulted in increased density of the treated fill deposits. To illustrate the effects of ground improvement, Fig. 3 comparatively shows the pre-earthquake penetration resistance at two sites in the northwest part of Port Island. The sites are located in close proximity to each other, at a distance of about 130 m (Fig. 2a). The VA-site has untreated fills, as originally deposited in the reclamation, and shows low SPT resistance with a blow count of 5 to 10 throughout the depth of the fill deposit. On the other hand, the reclaimed fills at the PH-site have been densified by the rod compaction method down to a depth of 15 m resulting in significantly increased penetration resistance with *N* values somewhere in the range between 20 and 40 blow counts.

After the earthquake, comprehensive laboratory studies have been conducted to investigate the liquefaction resistance of the fill soils. Results from a series of cyclic undrained tests on undisturbed samples of Masado soils are summarized in a typical plot correlating the cyclic stress ratio and the number of cycles required to achieve 5 % double-amplitude strain, shown in Fig. 4. The high-quality undisturbed samples have been recovered by the in-situ ground freezing technique from both untreated and densified fills. In addition to the laboratory tests, SPT measurements have been carried out at the sampling sites thus enabling us to associate an SPT blow count to each of the undisturbed samples. By grouping the samples according to their normalized SPT resistance (N_1 value), it was then possible to establish average liquefaction-strength curves for different blow counts, as depicted by the dashed lines in Fig. 4. Note that the significant variation in the SPT resistance is owing to the fact that samples of both untreated and densified fills are included in the plot. It is evident that the liquefaction strength of untreated fills (N_1 = 7.5-10) is low and that the cyclic strength increases with the SPT resistance. Clearly, the densification resulted in increased liquefaction resistance of the fills treated by ground improvement measures. It is essential to recognize, however, that in spite of the high gravel content, the liquefaction resistance of Masado soils is generally similar to that of clean sands.

3 LIQUEFACTION OF INLAND FILLS

The violent shaking of the main shock of the quake caused widespread liquefaction in the reclaimed lands. The liquefaction resulted in large cyclic displacements of the inland fills whereas very large

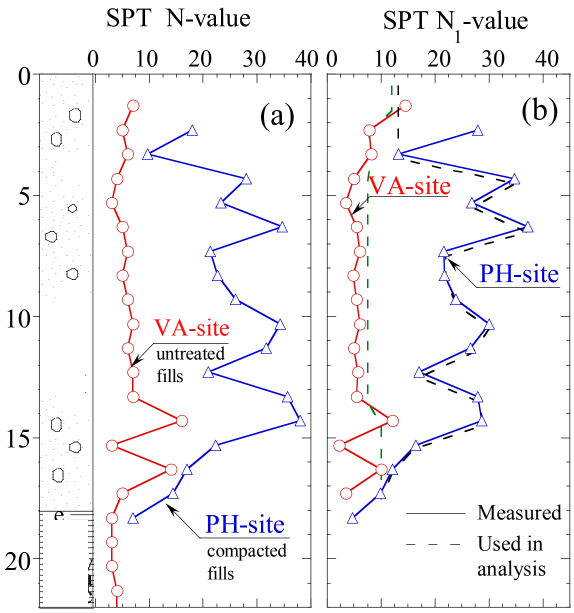

Figure 3. SPT resistance of untreated (VA-site) and compacted fills (PH-site); (a) *N* values; (b) N_1 values

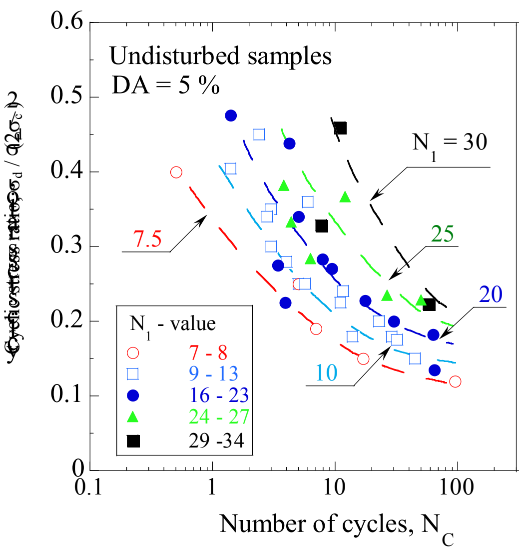

Figure 4. Undrained cyclic strength of undisturbed samples of reclaimed soils

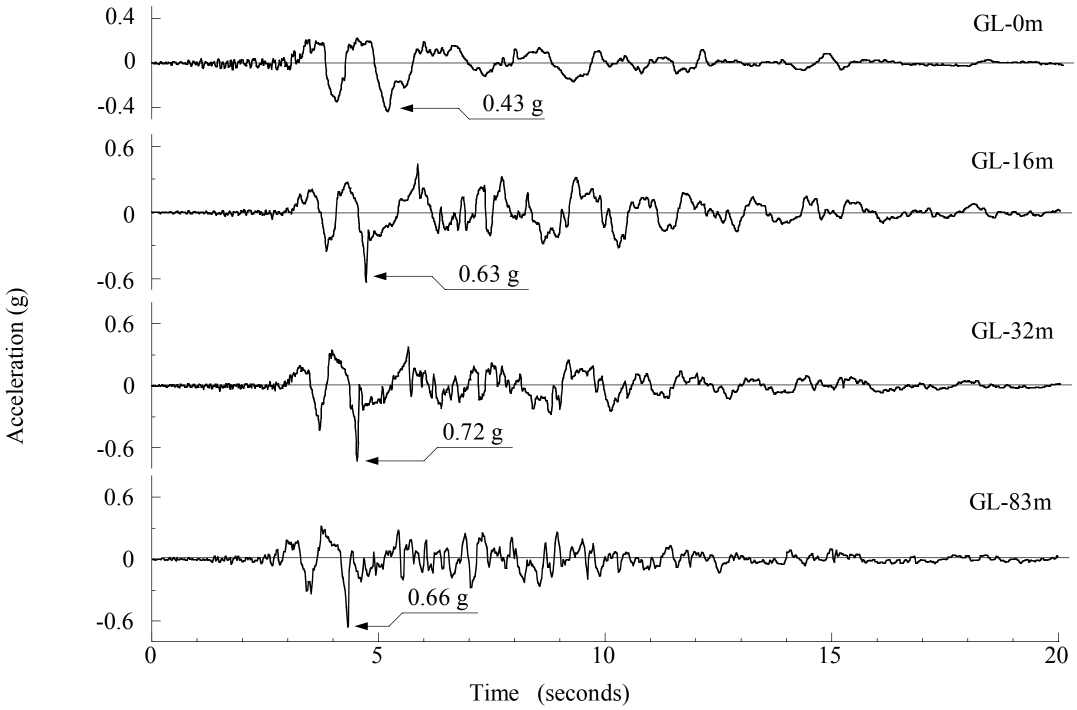

Figure 5. NW-SE ground accelerations recorded at the down-hole array site in Port Island (VA-site)

permanent displacements due to spreading of liquefied soils occurred within a zone of about 200 m from the revetment line. Features of liquefaction in areas that were not affected by lateral spreading are first described by closely examining the response of the two sites introduced above.

The massive liquefaction of untreated fills was manifested by a thick layer of sand and water littering wide areas of Port Island (Fig. 2a) and an average settlement of the ground of about 30-40 cm. On the other hand, fewer signs of liquefaction and only scattered sand boils were observed in the areas of densified fills. This distinction is clearly displayed in Fig. 2a where most of the central part of the island which is seen free of sand boils, in fact coincides with the areas where ground improvement had been executed (Yasuda et al., 1996).

At the VA-site, ground accelerations induced by the main shock of the quake have been recorded by an array of down-hole accelerometers. The recorded NW-SE accelerations at the ground surface, 16, 32 and 83 m depth are shown in Fig. 5. The ground motion is characterized by a relatively small number of intensive cycles and very high peak accelerations of 0.6-0.7 g. The motion exhibited very pronounced directionality with the maximum shaking intensity being oriented approximately in the northwest-southeast direction. Clear signs of liquefaction at this site are apparent in the decrease in amplitudes, loss of high frequency content and elongation of the predominant period of the motion recorded at the ground surface.

Detailed effective stress analyses of the VA-site and PH-site (Cubrinovski et al., 2000) revealed significant differences between the untreated fills and densified fills both in the extent and consequences of liquefaction. As illustrated in Fig. 6 where results of the analyses are summarized, the untreated fills at the VA site completely liquefied below the water table (Fig. 6a) resulting in maximum shear strains of 3.5-4% (Fig. 6b) and settlement of about 25 cm (Fig. 6c). On the other hand, the densified fill layer at the PH-site liquefied only in the deep part below 8 m depth and exhibited smaller peak shear strains of about 2 %. A simplified estimate of the volumetric strains resulting from dissipation of excess pore pressures revealed a settlement of about 8 cm which is in agreement with that actually observed at the PH-site. Thus, the increased density of the improved fills prevented occurrence of liquefaction in the shallow fills and limited the deformation at larger depths of the deposit. In accordance with the ground deformation as above, the relative displacement within the Masado layer

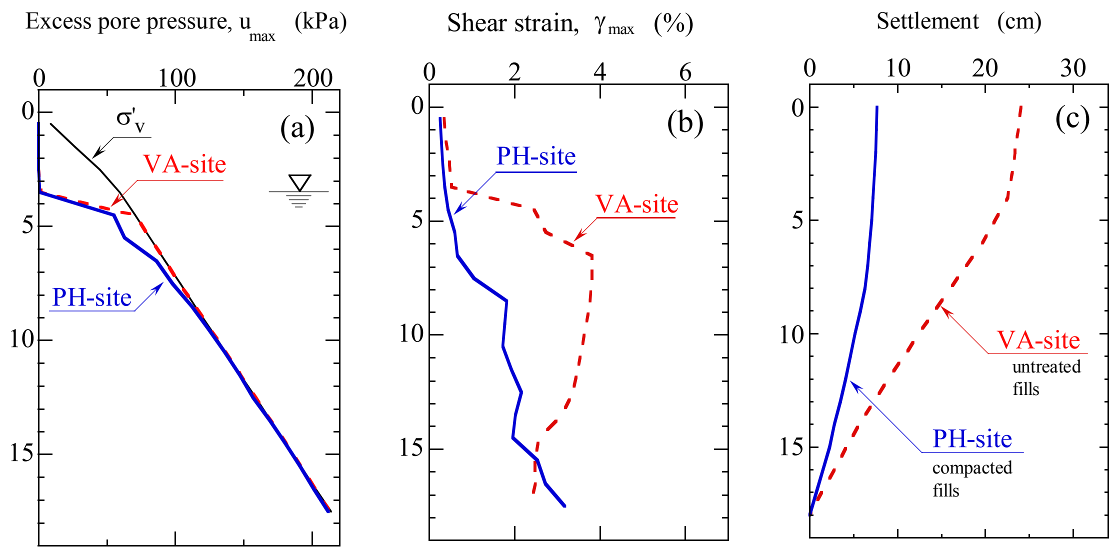

Figure 6. Computed maximum responses of untreated fills (VA-site) and compacted fills (PH-site): (a) excess pore pressures; (b) shear strains; (c) settlements

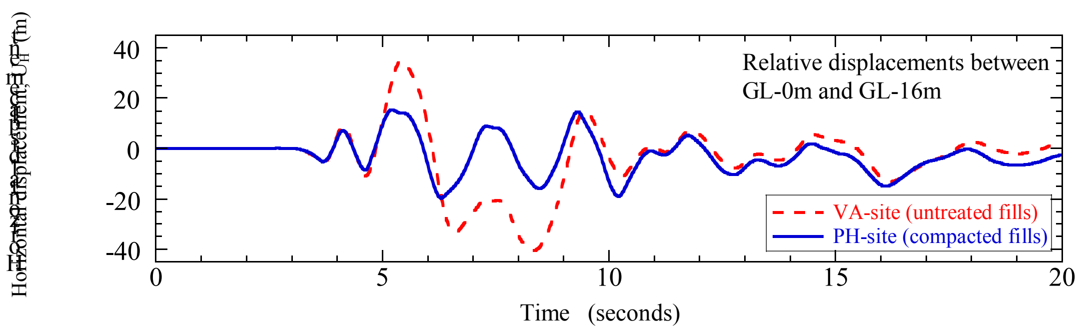

Figure 7. Computed relative displacements of the fill layers at the VA-site and PH-site

was found to be much smaller in the densified fills as compared to that of the untreated fills. As shown in Fig. 7, the peak horizontal displacement of the ground reached about 20 cm at the PH-site whereas it was as large as 40 cm at the VA-site.

4 LATERAL SPREADING

The quay walls in the Kobe harbor are gravity-type walls constructed of large caisson boxes in-filled with local soils. The height of the walls is typically between 8 and 18 m, and the average height to width ratio (H/B) may be roughly approximated as $H = 1.6B$ (Fig. 8a). Since the original seabed consisted of soft clay, this layer has been replaced with sand in order to provide bearing stratum for the heavy caisson walls. The thickness of the foundation sand (T_H) is quite large and ranges between 5 and 25 m. The materials behind the walls as well as those used as a foundation bed were Masado soils.

During the earthquake, many of the caisson walls moved up to several meters towards the sea and equally large magnitude of lateral spreading occurred in the backfill soils behind the walls. The permanent ground displacements due to spreading of liquefied soils extended inland as far as 200 m from the revetment line. In order to delineate features of movements of the quay walls and distortion of the ground in the backfills, measurements were made of the width of cracks and vertical offsets on the ground surface along alignments in the direction perpendicular to the revetment line (Ishihara et al., 1997). Typical output of such measurements is displayed in Fig. 9 for the cross section M-5 in

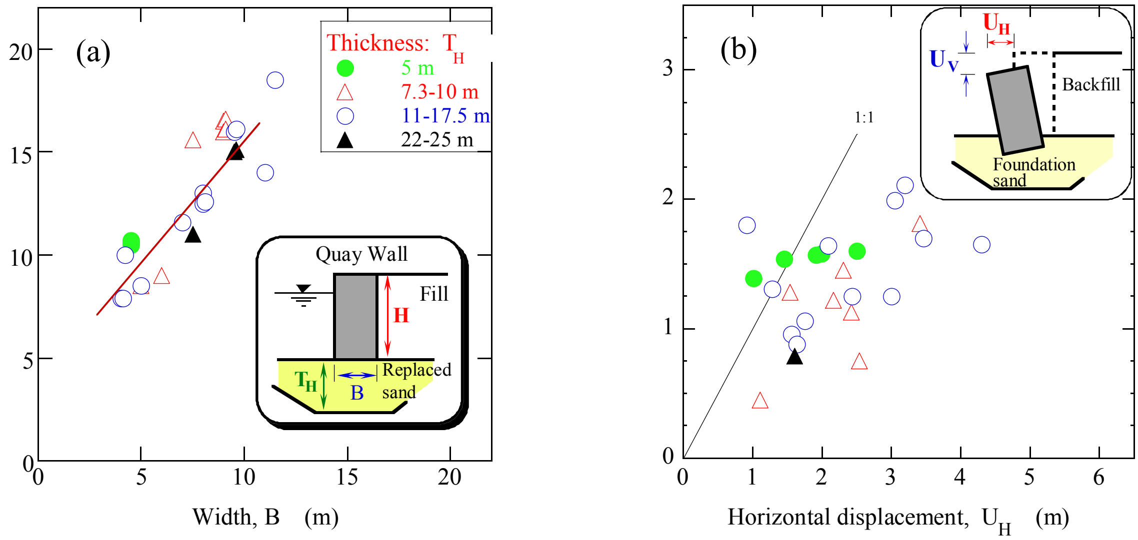

Figure 8. Quay walls in the port area of Kobe: (a) geometry features; (b) observed permanent displacements

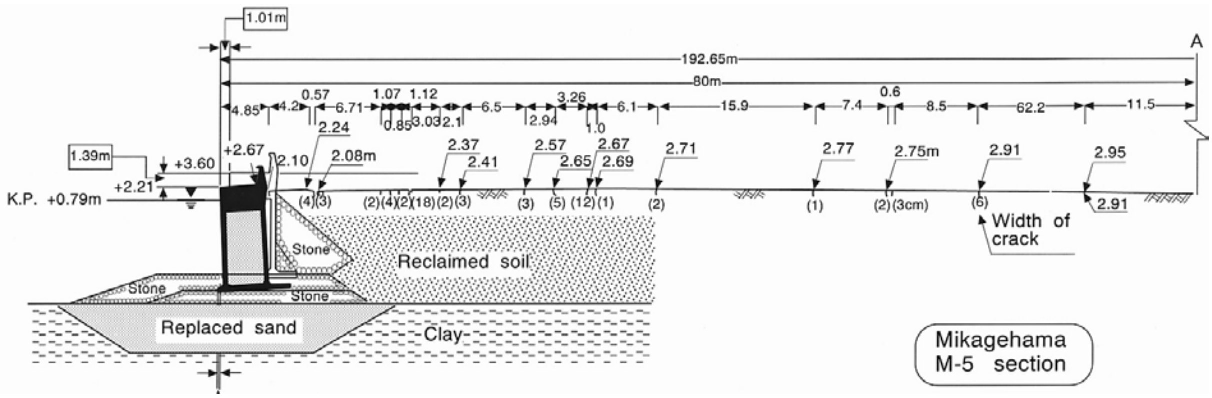

Figure 9. Permanent displacements of quay wall and ground at section M-5 in Mikagehama Island (Tank TA72)

Mikagehama Island where permanent lateral displacement of the wall of 1.01 m and settlement of 1.39 m were measured. The ground displacement is shown to have extended inland as far backwards as 192 m. Results of measurements as above along a number of sections at several man-made islands are summarized below.

The measured permanent displacements of the caissons are plotted in Fig. 8b. The horizontal displacements of the walls are very large and mostly in the range between 1 m and 4 m; the vertical displacements are generally around 1.0-2.0 m. In addition to the large inertial forces acting on the quay walls, the liquefaction of the backfill soils and foundation sand are considered to be key factors contributing to the large permanent movement of the walls. Detailed discussion on the displacement patterns of the quay walls including effects of dimensions (weight) of the caissons, thickness of foundation sand and ground treatment can be found in Ishihara and Cubrinovski (1999).

Ground distortion measurements as exemplified in Fig. 9 were summarized in plots depicting the permanent lateral ground displacement as a function of the distance inland from the waterfront. Figure 10a shows such plot for the ground displacements measured along 20 alignments of caisson walls at Port Island, Rokko Island and Fukaehama. It is seen that the permanent ground displacements due to spreading extend on average about 150 m inland from the quay walls. The magnitude of lateral spreading sharply increases at a distance of about 50 m from the revetment line and reaches its maximum at the quay walls. The excessive ground deformation as above is of engineering significance as

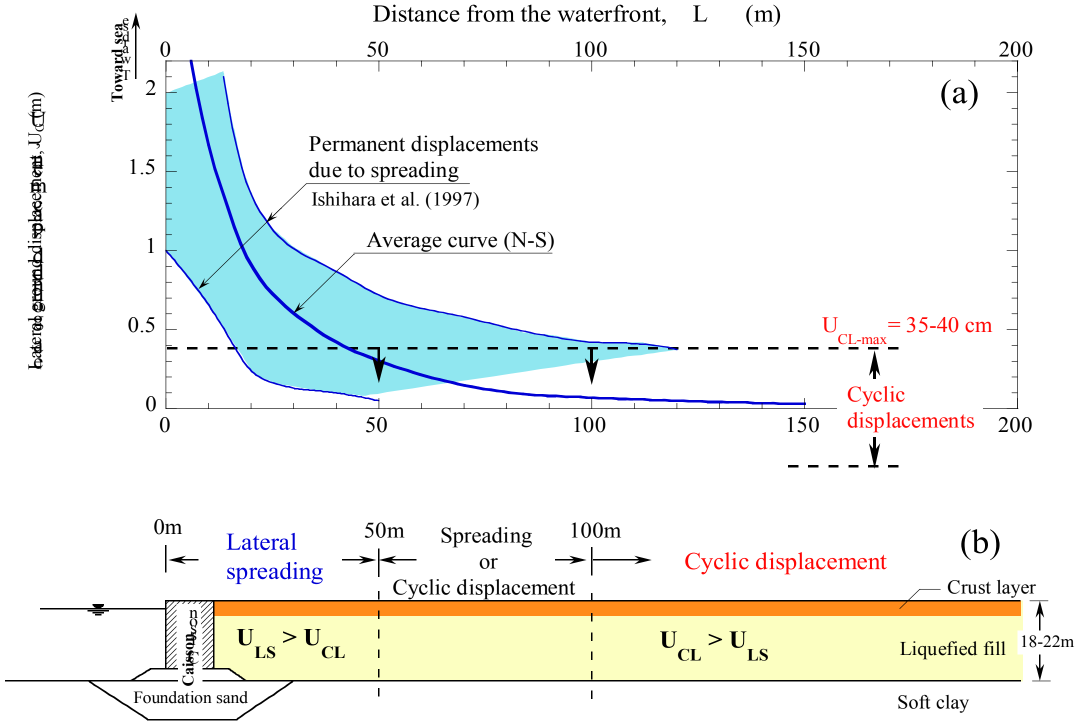

Figure 10. Characteristics of lateral ground displacements: (a) horizontal displacements as a function of the distance from the waterfront; (b) zoning for predominant influence of spreading or cyclic displacements

it defines the spatial extent of ground distress to which due consideration may need to be given in relation to the design of foundations. In this context, it is interesting to compare how the ground displacements due to lateral spreading correlate with those induced by the cyclic loads. As indicated in Fig. 10a and discussed earlier, the horizontal cyclic displacements of liquefied untreated fills were about 35-40 cm. Thus, with respect to the magnitude and features of lateral ground displacements we can distinguish three different zones in the waterfront area, as depicted in Fig. 10b. In the zone within a distance of about 50 m from the quay walls, the permanent ground displacements due to lateral spreading are significantly greater than the liquefaction-induced cyclic displacements. Conversely, the cyclic displacements are larger than the spreading displacements at distances greater than 100 m inland from the walls. In the intermediate range of approximately 50 to 100 m from the revetment line, either cyclic or lateral spreading displacements were predominant.

5 PERFORMANCE OF PILE FOUNDATIONS

High-rise buildings, storage tanks and bridge piers in the reclaimed areas of Kobe are typically supported on pile foundations, with the upper Pleistocene deposit serving as a bearing stratum for the piles. The concrete piles are either pre-cast concrete piles, mostly with diameters of 35-80 cm, or cast-in-place concrete piles with diameters typically over 100 cm. Detailed field inspections of piles conducted after the earthquake revealed that a large number of piles in the liquefied fills suffered serious damage or failure (JGS, 1998; Tokimatsu and Asaka, 1998). We will illustrate the role of liquefaction-induced ground displacements in the damage to piles by closely examining two well-documented case histories.

An oil-storage tank (TA72) supported on piles is located in Mikagehama Island, about 25-40 m inland from the revetment line at section M-5 shown in Fig. 9. Cross-sectional and plan views of the tank and its foundation are shown in Figs. 11a and 11b respectively. The tank, with a diameter of approximately 15 m and capacity of 2450 kl, is supported on 69 PHC (pre-stressed high-strength concrete) piles which are 23 m long and 45 cm in outer diameter. As indicated in Fig. 11, sand compaction

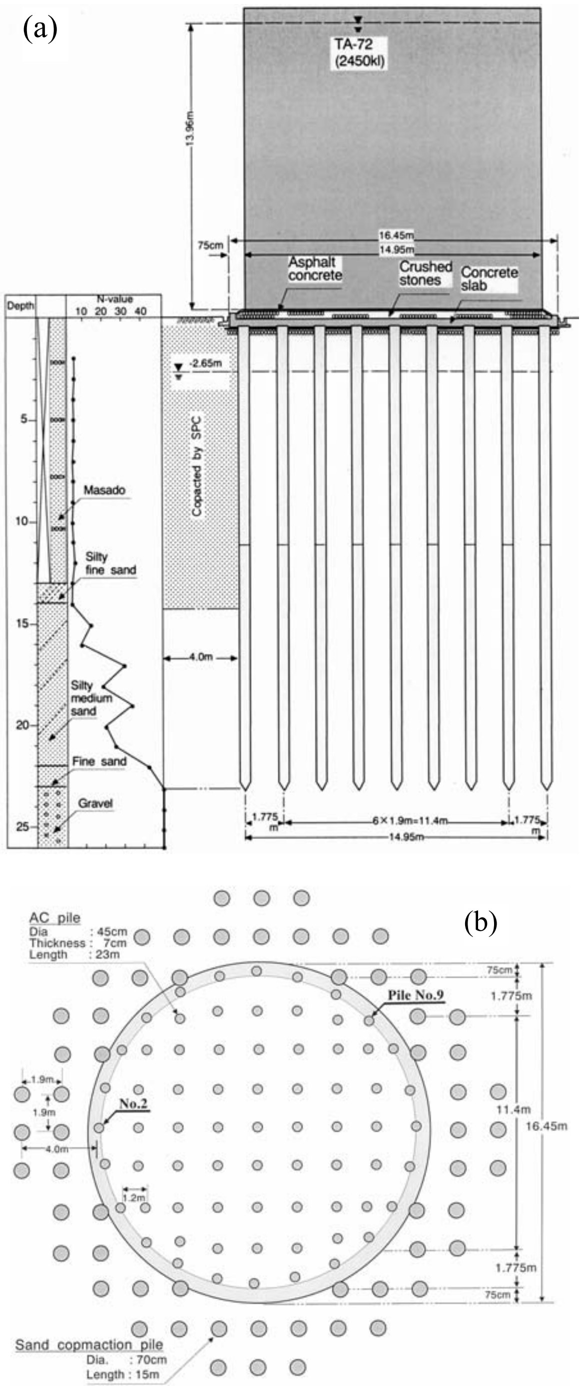

Figure 11. Mikagehama Tank TA72: (a) cross-sectional view; (b) plan view of foundation

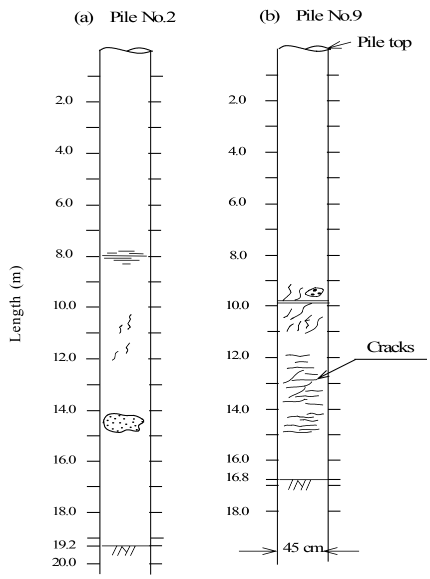

Figure 12. Observed damage to piles of Tank TA72

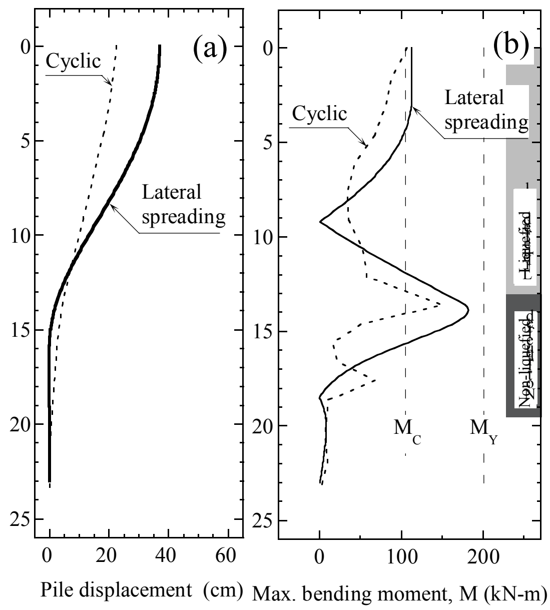

Figure 13. Computed pile response (TA72): (a) lateral displacements; (b) bending moments

piles have been installed around the perimeter of the foundation to improve the foundation soil. During the earthquake, extensive liquefaction occurred in the reclaimed fills resulting in a distortion of the ground in the backfills, as depicted in Fig. 9. It was estimated that at a distance corresponding to the location of the tank, the permanent ground displacement toward the sea was around 30-60 cm. Following the earthquake, two of the piles were investigated by lowering a bore-hole camera into the hollow cylindrical piles throughout the depth. The outcome of the bore-hole camera recordings is summarized in Fig. 12 where observed distribution of cracks is shown for the examined piles. The piles developed multiple cracks and suffered largest damage at a depth of approximately 12 to 14 m which corresponds to the depth of the interface between the liquefied fills and underlying non-liquefied layer.

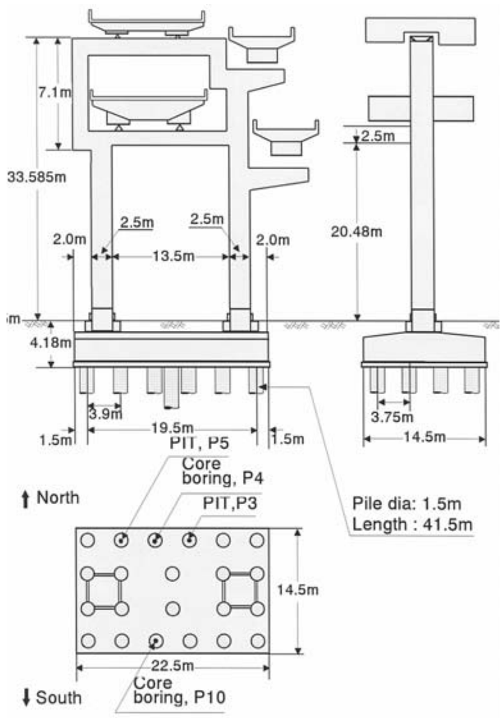

Figure 14. Cross sections of Uozakihama Pier 211 and plan view of its foundation (Hanshin Highway Authority, 1996)

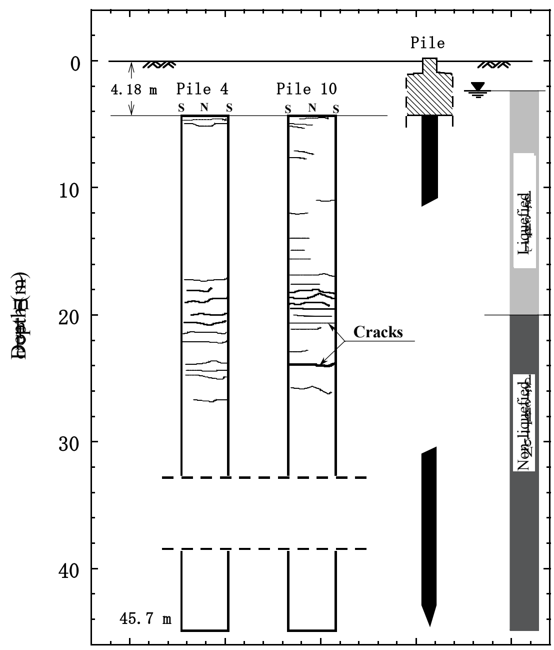

Figure 15. Observed damage to piles of Pier 211 (Hanshin Highway Authority, 1996)

To examine the pile behavior in more details and distinguish the response induced in two different loading phases, two analyses were conducted using dynamic effective stress analysis of soil-pile-tank-fluid system and pseudo-static analysis of a simplified soil-pile model respectively (Cubrinovski and Ishihara, 2001). The former analysis was used to evaluate the pile response in the course of cyclic loading and development of liquefaction while the latter served to estimate the pile response induced by the lateral spreading. Results of the analyses are summarized in Figs. 13a and 13b where maximum pile displacements and bending moments are shown respectively. At the time of the earthquake, the tank was filled to about 20 % of its capacity, and therefore, the impulsive pressure and sloshing effects from the oil were insignificant. Thus, even during the cyclic loading, the inertial loads were relatively small as compared to the kinematic loads on the piles induced by the cyclic ground displacements. The difference in the magnitude of the cyclic response and that induced by the lateral spreading seen in Fig. 13 merely reflects the difference in the respective ground displacements, which were about 25 cm in the cyclic phase and 45 cm in the lateral spreading analysis. Results of the analyses suggest that the piles were partially damaged during the cyclic liquefaction and that larger damage was inflicted during the subsequent lateral spreading of the liquefied soils.

Deleterious effects of lateral spreading were also observed in the performance of large-diameter piles supporting bridge piers in proximity to revetment lines. A typical example of such damage was seen in Pier 211 of the Hanshin Bay Route No. 5 which is located in the southernmost part of Uozaki-hama Island. Side views of Pier 211 and plan view of its foundation are shown in Fig. 14 (HHA, 1996). The pier is supported on 22 cast-in-place reinforced concrete piles, 1.5 m in diameter. The damage to the piles summarized from bore-hole camera recordings of two inspected piles is shown in Fig. 15. The damage at the pile head was supposedly caused by the inertial forces from the superstructure during the intense shaking. By and large however, the cracks were predominantly observed at depths corresponding to the interface between liquefied and non-liquefied layers. This type of damage was typically observed in piles located near the waterfront and may be attributed to the large ground displacements induced by the spreading of liquefied soils (Ishihara and Cubrinovski, 1998).

6 CONCLUDING REMARKS

There are several distinctive features of the liquefaction during the 1995 Kobe earthquake. Certainly, the extent of the liquefaction in which thick fill deposits massively liquefied over large areas of reclaimed lands was unprecedented by itself. Two important features are to be emphasized, however, in relation to the subsequent revision of liquefaction-evaluation procedures based on the Kobe event. One is the grain-size composition of the fill soils and especially their high gravel content. Prior to the Kobe earthquake, such gravelly soils have generally been considered of low susceptibility to liquefaction. The other is related to the characteristics of the ground motion, which in the case of the Kobe earthquake was characterized by a relatively small number of cycles, but of very high intensity, thus emphasizing the need to consider shock-type waves in the liquefaction evaluation.

The liquefaction-induced ground displacements both due to cyclic loads and especially those due to spreading of liquefied soils were very large. The quay walls moved up to several meters towards the sea and equally large magnitude of lateral spreading occurred in the backfill soils. Permanent lateral ground displacements due to spreading were predominant within a zone of about 50 m from the quay walls and extended on the average about 150 m inland from the revetment line. On the other hand, cyclic displacements of about 35-40 cm characterized the response of the inland fills at distances greater than 100 m from the quay walls. Delineation of excessive ground deformation as above is of engineering significance as it defines the spatial extent of ground distress and identifies relevant ground displacements to which due consideration needs to be given in relation to the design of foundations. Field observations and numerical analyses clearly indicate that ground improvement measures reduced both the extent of the liquefaction and consequent ground deformation.

The excessive lateral ground displacements due to spreading of liquefied soils had significant impact on the damage to pile foundations. In particular, the damage to the piles at larger depths near the interface between the liquefied soil and underlying non-liquefied layer can be attributed to the lateral ground displacements. Prior to the Kobe earthquake, however, effects of lateral ground displacements had been, by and large, either ignored or crudely approximated in the seismic codes for piles. For these reasons, the kinematic loads on piles resulting from lateral ground displacements received particular attention after the earthquake. Significant efforts have been made over the past eight years to investigate the mechanism of lateral spreading and establish procedures for assessment of piles including benchmark full-scale tests on piles using large-scale shake table.

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