INVESTIGATION OF LIQUEFACTION AT RECLAIMED LAND IN JAPAN DURING THE 1995 HYOGOKEN-NAMBU EARTHQUAKE

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SUMMARY

Numerous instances of liquefaction occurred in Kobe Port Island and other areas of reclaimed land in Japan during the 1995 Hyogoken-Nambu earthquake. However, no occurrences of sand boils resulting from liquefaction were found in almost all the improved ground areas. To investigate liquefaction behavior of ground for an improved site and an unimproved site during the earthquake, seismic response analysis and simplified liquefaction evaluation at both sites were carried out in this study. The seismic response analysis used SHAKE, code of equivalent linear analysis. The input motion used for this analysis was the strong motion vertical array record obtained at Kobe Port Island, and the result of this analysis was compared with strong motion observation. Then, the authors confirmed that the equivalent linear analysis is capable of simulating a seismic response of ground until liquefaction occurs. The simplified liquefaction evaluation used maximum response shear stress computed by SHAKE and cyclic shear strength determined by the cyclic undrained triaxial test on undisturbed frozen specimens obtained from improved and unimproved grounds.

Results of the simplified liquefaction evaluation were generally consistent with the facts that damage occurred at both ground surfaces. The authors concluded that liquefaction occurred at a deep layer not only at the unimproved site but also at the improved site, where the seismic response above the liquefaction layer was relatively small because the liquefaction layer acted like a seismic isolation layer.

INTRODUCTION

Liquefaction of ground occurred at many coastal areas along Osaka Bay from Kobe to Osaka during the 1995 Hyogoken-Nambu earthquake in Japan. In particular, liquefaction occurred nearly all over Kobe Port Island (KPI), which was reclaimed by decomposed granite soil, *Masado*, while settlement of 70 cm was observed at some areas. However, there were some parts of KPI that were improved through the sand drain method or the rod compaction method, and no sand boils resulting from liquefaction were found in almost all the improved areas [Kobe City Development Bureau, 1995].

To investigate liquefaction behavior of ground for an improved site and an unimproved site at KPI during the earthquake, seismic response analysis using strong motion vertical array records obtained at KPI [Kobe City Development Bureau, 1995] and simplified liquefaction evaluations at both sites were carried out in this study.

INVESTIGATION SITES AND OUTLINE OF DAMAGE

Investigation of liquefaction behavior of ground was carried out for the improved and unimproved sites in KPI. Figure 1 shows the investigation sites.

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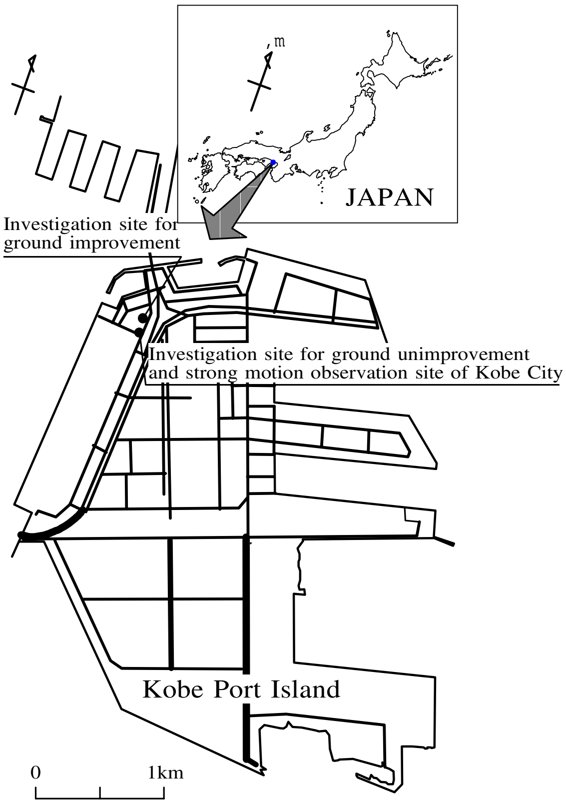


Fig.1 Investigation sites

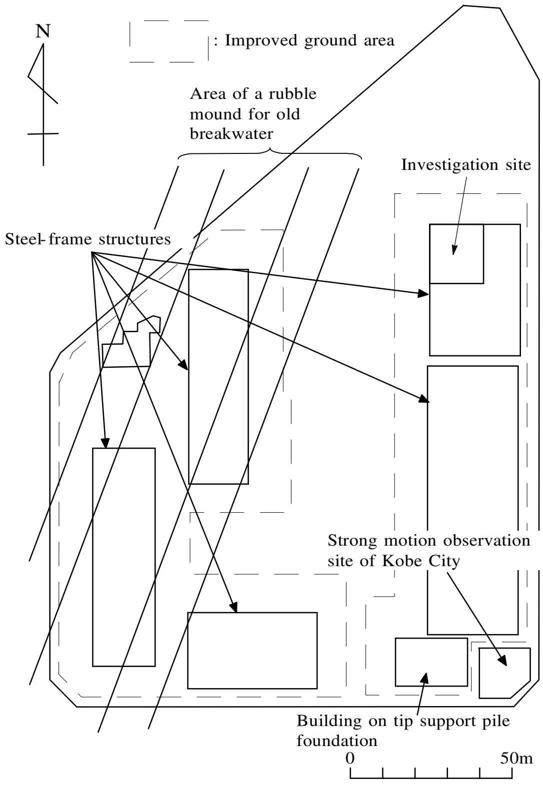


Fig. 2 Improved site [Geotechnical Research Collaboration Committee on the Hanshin-Awaji Earthquake, 1998]

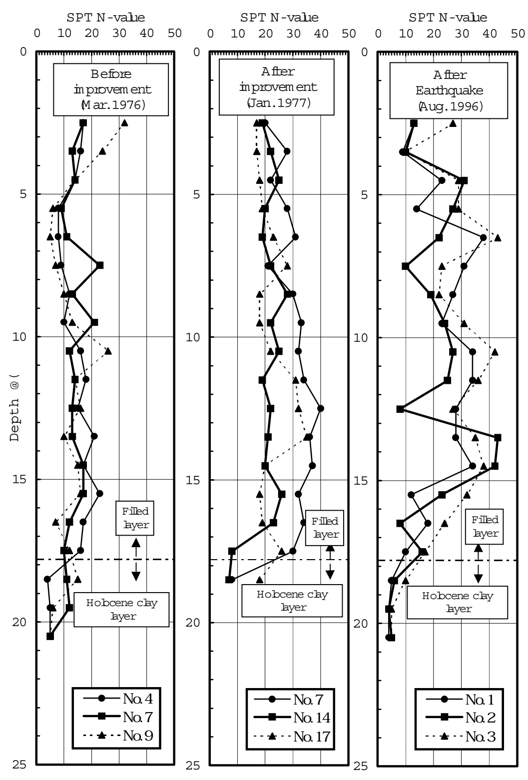


Fig. 3 Profiles of SPT *N* value at investigation site

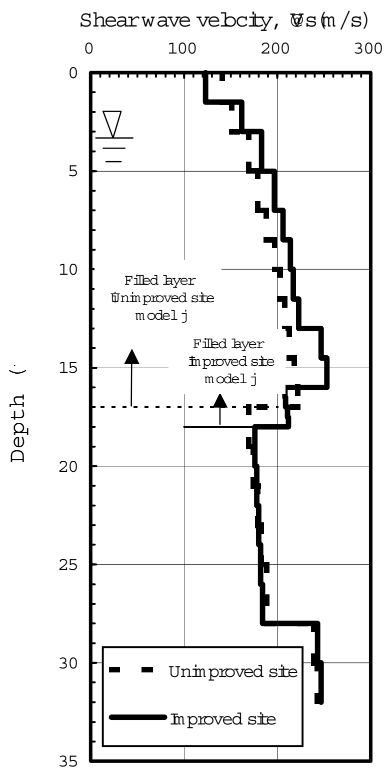


Fig. 4 Profile of shear wave velocity

Figure 2 shows the improved site. The improved site is located in the northwestern part of KPI and 130 m north of a point where Kobe City obtained strong motion vertical array records [Kobe City Development Bureau, 1995]. Ground shallower than about GL-18 m is a filled layer reclaimed by *Masado* and under this filled layer is a Holocene clay layer. The ground improvement for this filled layer had been performed by the rod compaction method in the areas of steel-frame structures built with individual footings and areas surrounding them. Figure 3 shows profiles of *N*-values before ground improvement in 1976, after ground improvement in 1977 and after the earthquake in 1996. It is found that *N*-values that were 10 - 15 before the improvement increased to 20 - 35 after the improvement and after the earthquake. Overhead cranes in the steel-frame structures except those located just above a rubble mound for the old breakwater were operable after the earthquake. In this area, no sand boils resulting from liquefaction were observed, but the ground surface sank approximately 30 cm compared with the building on the tip support pile foundations. This settlement is supposed to also include the settlement before the earthquake. Another investigation by Hamada [Hamada, Isoyama and Wakamatsu, 1995] shows the settlement was 10 cm and the lateral displacement of the ground surface was 57 cm as the difference before and after the earthquake in this neighborhood. Overhead cranes in the steel-frame structures located just above a rubble mound for the old breakwater could not be operated after the earthquake. In the unimproved areas surrounding the improved areas including the strong motion observation site of Kobe City, a lot of sand boils were observed, and the settlement was several tens of centimeters more than those in the improved areas [Geotechnical Research Collaboration Committee on the Hanshin-Awaji Earthquake, 1998].

The strong motion observation site of Kobe City was investigated as the unimproved site. The settlement and the lateral displacement near this unimproved site were 47 cm and 33 cm, respectively [Hamada, Isoyama and Wakamatsu, 1995].

ANALYSIS PROCEDURE

Seismic response analysis

Seismic response analysis was conducted by equivalent linear analysis using SHAKE [Schnabel, Lysmer and Seed, 1972]. Grounds for the improved and unimproved sites shallower than GL-32 m were modeled. The input base grounds were set at a depth of GL-32 m where the strong motion vertical array record was obtained during the earthquake. The input motion was an incident wave (2E) that was computed by equivalent linear analysis using the 1995 Hyogoken-Nambu earthquake motion records at GL-32 m and GL-83 m [Geotechnical Research Collaboration Committee on the Hanshin-Awaji Earthquake, 1998]. The direction of the input motion was N38.8W.

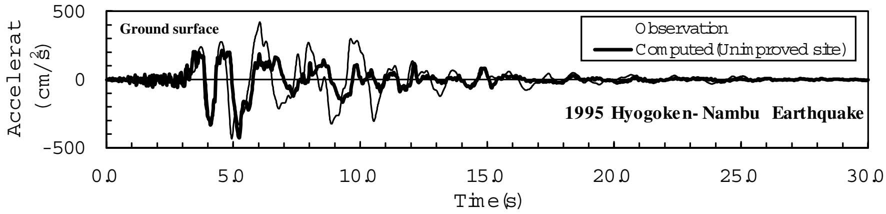
Figure 4 shows a profile of the shear wave velocities of the improved and unimproved site. These *Vs*-values were originally determined by P-S logging and adjusted slightly. It was noticed that *Vs*-values of the improved site at depths ranging from GL-16 m to GL-18 m were almost the same as those of the unimproved ground. This is because both P-S logging values by the downhole method and the suspension method and the *N*-values at this area were as small as at the unimproved ground.

Simplified liquefaction evaluation

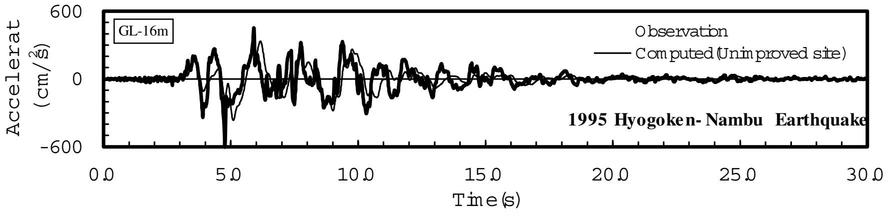
To investigate liquefaction behavior of ground, a simplified liquefaction evaluation was carried out using a method partially based on that specified in *the Design Specifications of Highway Bridges* in Japan [Japan Road Association, 1996]. The liquefaction resistant ratio F_L was the ratio of cyclic shear strength (liquefaction strength) to the maximum shear stress during an earthquake. The cyclic shear strength was determined by the cyclic undrained triaxial test on undisturbed frozen specimens obtained from the improved ground and from the unimproved ground at approximately 200 m south of the point of the improved investigation site [Hatanaka, Uchida and Ohara, 1997]. Because the layer of the improved site at depths ranging from GL-16m to GL-18 m was considered to be unimproved, the cyclic shear strengths at these depths were assumed to be the strengths of the unimproved ground. The maximum shear stress during the earthquake was computed by seismic response analysis as above.

RESULT OF SEISMIC RESPONSE ANALYSIS

Figure 5 shows observed acceleration time histories at the unimproved site where the strong motion records were observed. They are compared with computed acceleration time histories at the unimproved site in these figures. Figure 5 (a) is at the ground surface and Figure 5 (b) is at GL-16 m. The maximum acceleration of the observation is 426 cm/s^2, and the computed maximum acceleration is 434 cm/s^2. The computed maximum acceleration corresponds with the observation as shown in Figure 5 (a). In addition, the amplitudes and phases of their acceleration time histories correspond until the end of

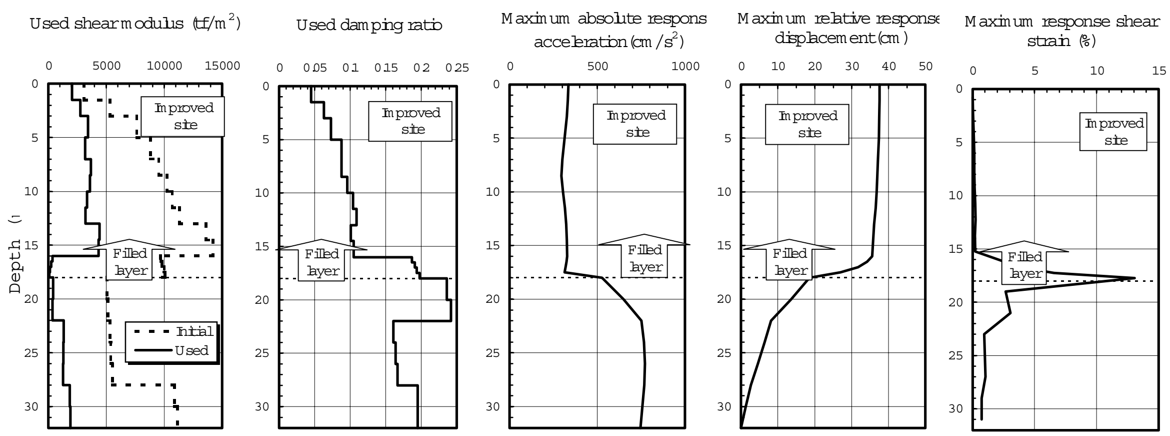


(a) Ground surface

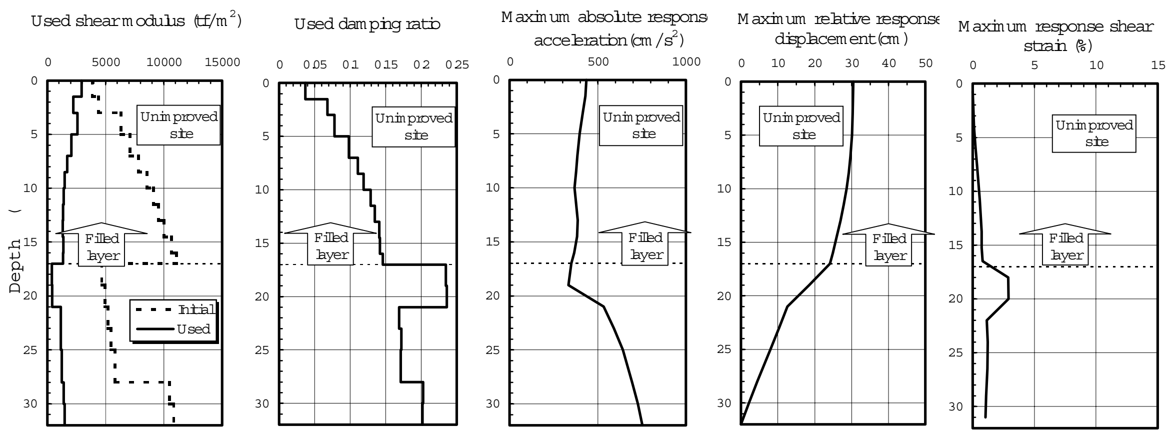


(b) GL-16m

Fig. 5 Acceleration time histories at the unimproved site



(a)Improved site



(b)Unimproved site

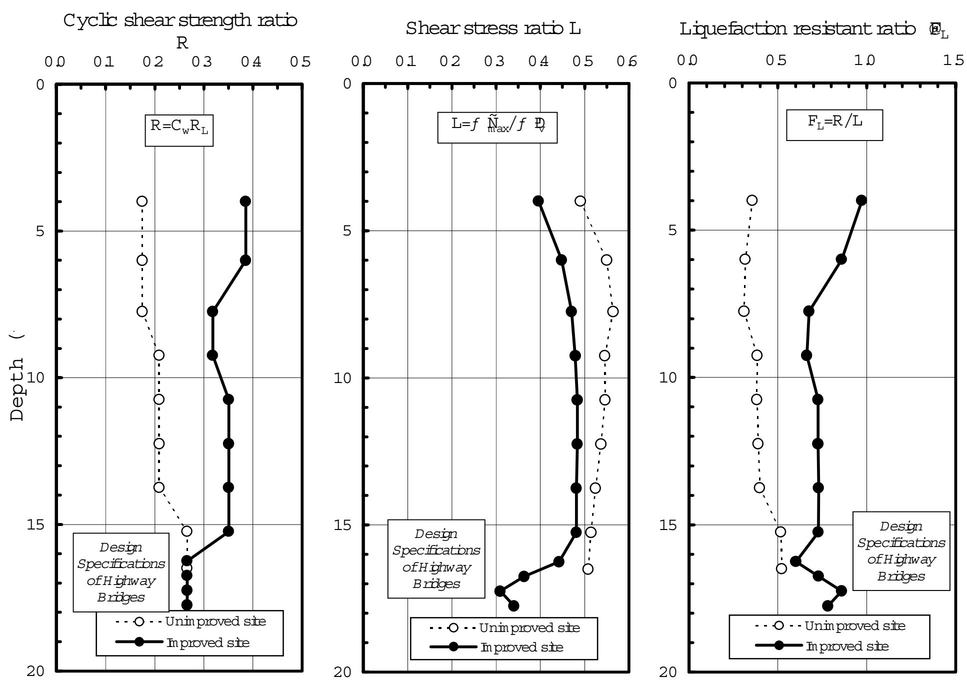
Fig. 6 Profiles of used shear modulus with initial shear modulus, used damping ratio, maximum absolute response acceleration, maximum relative response displacement and maximum shear strain for the improved site and the unimproved site

the second main wave when liquefaction occurs. At GL-16 m, the same results are recognized except a spike wave observed in the observation wave at around 5 seconds. Therefore, the authors consider that the equivalent linear analysis can simulate seismic response of ground until liquefaction occurs.

Figure 6 shows profiles of used shear modulus with initial shear modulus, used damping ratio, maximum absolute response acceleration, maximum relative response displacement and maximum response shear strain which were computed by SHAKE. Figure 6 (a) is for the improved site. Shear strains are as large as 3% at depths ranging from GL-19 m to GL-21 m of the Holocene clay layer. The maximum is significantly large at 13% from GL-16 m to GL-18 m, the lowest level of the filled layer. The *N*-values and shear wave velocities of this level were lower than those of the upper levels. Therefore, the response accelerations are almost the same at depths ranging from GL-22 m to GL-32 m, change abruptly into small acceleration at depths ranging from GL–18 m to GL-22 m, and are relatively small at about 300cm/s^2 in the improved filled layer above GL-18 m. Similarly, the response displacements suddenly become large at depths ranging from about GL-16 m to GL-20 m and are almost the same at the improved filled layer above GL-16 m. Figure 6 (b) is for the unimproved site. Shear strains are as large as 3% at depths ranging from GL-18 m to GL-20 m of the Holocene clay layer, the same as for the improved site. Maximum response accelerations change smoothly into small acceleration at depths ranging from GL-20 m to GL-32 m and almost the same at about 400 cm/s^2 at levels above GL-20 m. The maximum response acceleration of the unimproved site at the surface is larger than that of the improved site.

RESULT OF SIMPLIFIED LIQUEFACTION EVALUATION

Figure 7 shows the result of simplified liquefaction evaluation for the improved site and for the unimproved site. Figure 7 (a) through Figure 7 (c) show the cyclic shear strength ratio, shear stress ratio during an earthquake and the liquefaction resistant ratio F_L . R_l is a cyclic shear strength ratio determined by the cyclic undrained triaxial test. C_w is a corrective coefficient determined by the effect of the difference between the triaxial condition and the simple shear condition, effect of multi-direction shear, and effect of irregular seismic waves including the property of increasing ductility with an increase in the cyclic strength for the shock-wave type. The cyclic shear strength for the improved ground is hence larger than that for the unimproved ground. The shear stress ratio during an earthquake is the ratio of the maximum response shear stress, computed by SHAKE, to the effective overburden pressure. The shear stress ratios for the improved site are smaller than those for the unimproved site at all layers. Therefore, the liquefaction resistant ratios F_L for the improved site are larger than those for the unimproved site.



(a)Cyclic shear strength **(b)Shear stress ratio during an earthquake** **(c)Liquefaction resistant ratio**

Fig. 7 Result of simplified liquefaction evaluation

The liquefaction resistant ratios F_L are less than 0.5 at all layers for the unimproved site. It is consistent with the facts that many sand boils were observed and that more than several tens of centimeters of settlement occurred in KPI. On the other hand, for the improved site, the liquefaction resistant ratios F_L are almost 1.0 at GL-4 m and less than 0.75 at levels below GL-7 m. This result indicates that liquefaction occurred at levels below GL-4 m to GL-7 m not only at the unimproved site but also at the improved site. It is also consistent with the facts that the ground surface sank approximately 30 cm compared with the building on the tip support pile foundations, that the overhead cranes in steel-frame structures located just above a rubble mound for the old breakwater could not be operated after the earthquake, and that the settlement was 10 cm and the lateral displacement of the ground surface was 57 cm as the difference before and after the earthquake in the neighborhood, at the investigation site. However, though liquefaction occurred at levels deeper than GL-6 m at the improved site, almost no damage was observed on the ground surface. The authors consider the reasons for this were that approximately 6 m of a non-liquefied layer existed at the ground surface and the thickness of this non-liquefied layer was enough to resist cracking and differential settlement. In addition, the authors suppose that the seismic response above the liquefaction layer would be relatively small because the liquefaction layer acted like a seismic isolation layer.

CONCLUSIONS

The authors carried out seismic response analysis by equivalent linear analysis and simplified liquefaction evaluation at the improved and unimproved sites in Kobe Port Island to investigate liquefaction behavior of ground for both sites, using the strong motion records obtained during the 1995 Hyogoken-Nambu earthquake and using the cyclic shear strength determined by undisturbed samples. The conclusions obtained from this study are summarized as follows:

(1) Acceleration time histories computed by the equivalent linear analysis, SHAKE, for the unimproved site corresponded with the observation records until liquefaction occurred.

(2) The maximum response acceleration at the ground surface for the improved site was smaller than that for the unimproved site, because large shear strain for the improved site occurred at the lowest part of the filled layer, which was supposed to be the unimproved layer.

(3) Results of the simplified liquefaction evaluation were consistent with the facts that damage occurred at both ground surfaces of the improved and unimproved site.

(4) The authors considered that liquefaction occurred at a deep level not only at the unimproved site but also at the improved site.

ACKNOWLEDGMENTS

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