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LIQUEFACTION OF IMPROVED GROUND IN PORT ISLAND AND ITS EFFECT ON VERTICAL ARRAY RECORD

Nozomu YOSHIDA[1]

SUMMARY

Effect of remedial measures against liquefaction on the ground behavior during an earthquake is investigated through the case study at Port Island during the 1995 Hyogoken-nambu earthquake. The site is located just beside the downhole strong motion observation station and was improved by rod compaction. No significant ground deformation was observed during the earthquake. Since this site is very close to the station (the distance is within 10 m), the behavior may affect the vertical array records. Effective stress dynamic analysis is conducted in which material property is evaluated based on the detailed soil test at the improved and unimproved sites. The existence of the improved ground is shown to affect the vertical array record, but because of following several reasons, the effect is not significant because peak response is not affected. Liquefaction occurred in the lower part of improved ground; earthquake motion is filtered significantly by the nonlinear behavior of the soft marine clay layer beneath the fill; bottom of the fill is not improved; liquefaction and peak response occurred at first predominant wave

INTRODUCTION

Significant liquefaction was observed in the fill area in Kobe City, Japan, and vicinity during the 1995 Hyogoken-nambu earthquake. At the same time, this earthquake proved the effectiveness of the remedial measures against liquefaction. The settlement of the ground depended on the degrees of improvement of the ground; settlement is smaller as improvement becomes more heavy [Yasuda et al., 1995].

Liquefaction in Port Island, a man made island in Kobe City, was also severe. All the quay walls displaced toward the sea for about 1 to 5 meters [Inagaki et al., 1995]. The entire island except improved region was covered by boiled sand and colored brown. Geotechnical Research Collaboration Committee on the Hanshin-Awaji Earthquake Disaster [1998] conducted detained in-situ and laboratory tests at a site in Port Island where the ground was improved by the rod compaction method. Evidence of liquefaction such as sand boil and ground crack was not observed at the site, which showed clear contrast against the unimproved ground nearby where the ground was covered by boiled sand and significant settlement was observed. Detailed investigation, however, showed that ground settlement of about 10 cm occurred during the earthquake. Residual horizontal displacement was also observed [Hamada et al., 1995]. This fact seems to be against the common sense that large ground settlement does not occur if liquefaction does not occur.

In order to clarify this inconsistency, the author conducted earthquake response analyses based on effective stress [Yoshida and Ito, 1999]. Liquefaction was found not to occur at the upper part of the improved region, but it occurred in the deep layers. This result can explain the observed phenomena. If liquefaction occurred in deep layer, the whole ground can settle without significant ground deformation near the ground surface.

[1] *Engineering Research Institute, Sato Kogyo Company Limited, Tokyo, Japan.*
E-mail: Nozomu.Yoshida@satokogyo.co.jp

The investigated site is located just neighboring to the downhole strong motion observation station; the nearest distance is within 10 meters. Considering that the thickness of fill is 18 meters, it is natural to consider that the existence of this improved region affected the vertical array records although many effective stress analyses have been done in this site under 1-dimensional condition and have shown to agree with observed records. In order to respond this question, the author conducted two-dimensional effective stress dynamic response analysis, which will be reported in this paper.

SITE AND BRIEF REVIEW OF PAST RESEARCH

Figure 1 shows liquefaction in Port Island. The ground was covered by sand boils [Hamada et al., 1995] except the places where remedial measures were made or there exist buildings and houses in which case sand boil cannot be seen. The investigated area of about $150\times150m^2$ is located near the northwest corner of Port Island. Figure 2 shows details in this site. This site was filled by Masado, decomposed granite, in 1969. Ground improvement was made by rod compaction method in 1977 in order to make package factories. Five one-storied factories (steel-framed structures used for warehouse and workshop) and a three-storied reinforced concrete control building were built. All factories are supported by the spread foundation, and the control building was supported by the pile foundation.

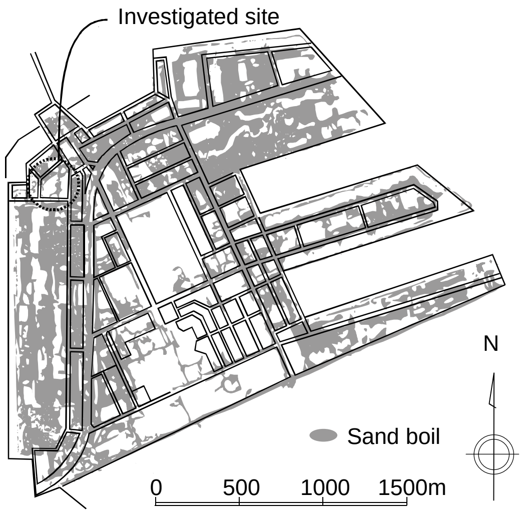


Figure 1: Liquefaction in Port Island and investigated site.

Detailed investigation was made at the northeast side of the site as shown in Figure 2. Many in-situ and laboratory investigations were made, in which freezing sampling and laboratory test using it were included. Figure 3 shows soil profiles at the strong motion observation station that is located at the southeast corner of the site. The thickness of fill is about 18 m. Soft Holocene layer called Ma13 with 10 m deep exists beneath the fill layer. Seismographs were installed at depths of GL, GL-16, GL-32 and GL-83 m. Figure 4 shows SPT *N*-value distribution at the improved site. The ground was improved by rod compaction method up to GL-16m. Although, as shown in Figure 3, the thickness of fill is 18 m, bottom 2 m was not improved because liquefaction was judged not to occur.

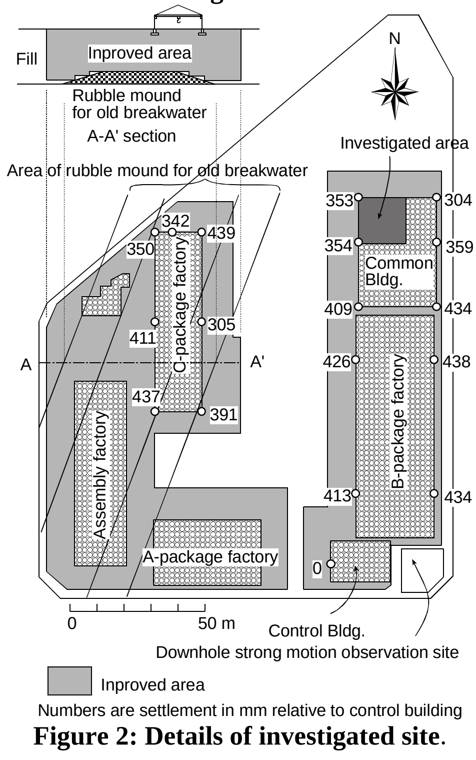


Figure 2: Details of investigated site.

Through the detailed investigation, several unusual phenomena were recognized. Firstly, differential settlement occurred at the C-package factory resulting in ceiling crane damage. Secondly, gap of about 30 cm was found to occur between the ground surface and entrance in the control building after the earthquake; this gap was not known before the quake. Since this building is supported by pile foundation, the gap should come from the settlement of the ground. Numbers in Figure 2 is relative settlement at several locations from the control building measured after the earthquake based on the design draft. These numbers include the settlement due to consolidation of marine clay as well as fill after reclamation, therefore cannot directly be compared with the gap. Several investigations, however, came conclusion that at least about 10 cm settlement occurred during the earthquake in the improved area.

Dynamic response analyses based on effective stress were conducted in order to know the behavior during earthquake as a part of the activities in the Geotechnical Research Collaboration Committee on the Hanshin-Awaji Earthquake Disaster (GRCC). Three computer codes, LIQUA, EFFECT and YUSAYUSA-2, were used in the analysis [GRCC, 1989; Yoshida and Ito, 1999]. All the analyses gave the same conclusion although the absolute

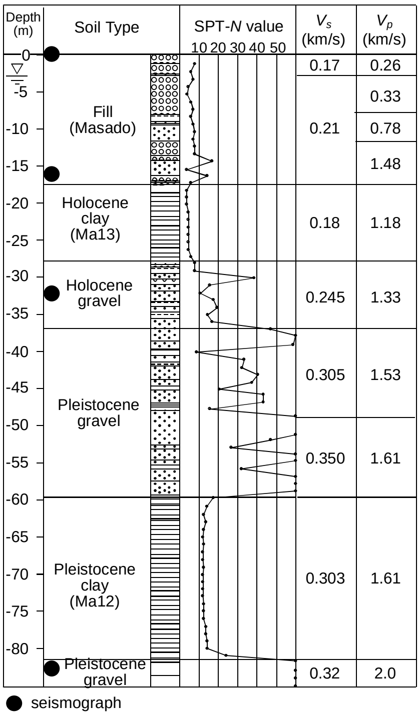


Figure 3: Soil profiles at downhole strong motion observation station

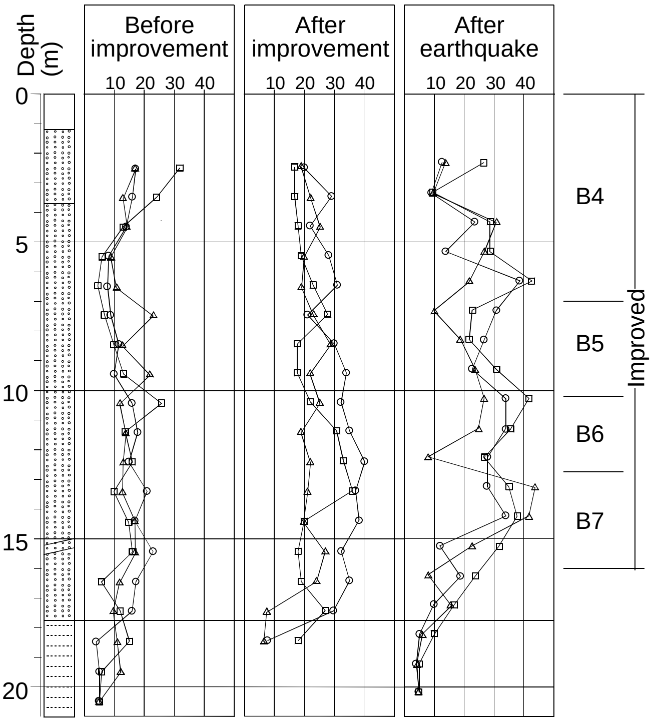


Figure 4: Soil profiles and SPT *N*-value before and after the improvement and after the earthquake.

This effect cannot be examined in the previous analyses because only one-dimensional analyses were conducted. In order to investigate the effect of the improved region on the earthquake behavior at strong motion observation station, at least 2-dimensional analysis is required. In this study, a general purpose code STADAS [Yoshida, 1993] is employed for the effective stress dynamic response analysis. The constitutive model developed by the authors [Yoshida and Tsujino, 1993; Yoshida et al., 1993] were a little modified and used.

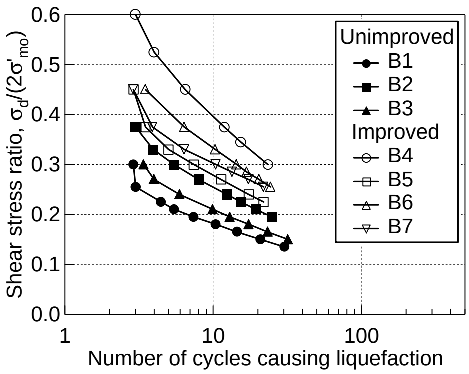


Figure 5: Liquefaction strengths at improved and unimproved sites

The ground shaking in the EW direction is investigate because, as seen in Figure 2, the station is in the shortest distance in this direction. This is different from previous analysis [Yoshida and Ito, 1999] in which ground shaking in NS directions was examined. Another difference from the previous analysis is input ground motion. Incident wave that was separated from the observed record by using the equivalent linear method was used in the previous analysis, whereas observed record at GL-32m was directly applied at the base of the analyzed region in this study. Other conditions are the same with previous analyses. The improved region is classified into four types as shown in Figure 4 based on SPT *N*-value distribution and result of other soil tests. Liquefaction strength of each layer is shown in Figure 5. This liquefaction strength is obtained in the laboratory test from the freezing sample, which is known to be different from the liquefaction strength from ordinary undisturbed as well as disturbed samples. Therefore, for the unimproved site, material property obtained from freezing sample is also used so as to make the analysis consistent. The investigated unimproved site is located about 70 m from the strong motion observation station [Suzuki et al., 1977]. The liquefaction strength of this site is also shown in Figure 5, and will be used as liquefaction strength at the strong motion observation site. The soil layers is divided into three layers, which is shown later in Figure 13

ONE DIMENSIONAL ANALYAIS AND DISCUSSION

In order to examine the applicability of the constitutive model, one-dimensional analysis is conducted first. Figure 6 compares computed acceleration time histories with observed records.

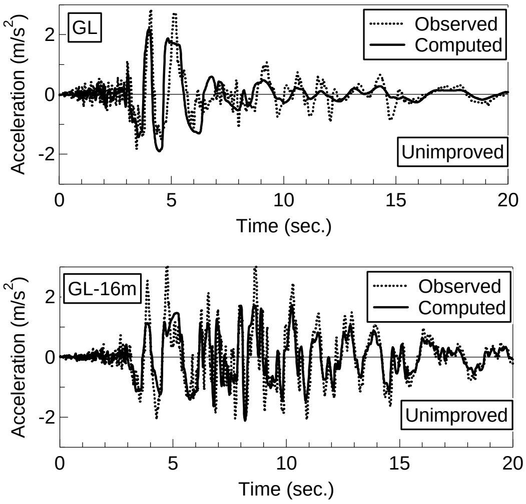


Figure 6: Acceleration at unimproved site

Generally, analysis agrees with observed record very well. Peak values, however, seem smaller than observed. This can be explained by considering the 2-dimensional effect on modeling. In the two dimensional analysis, since overburden stress and horizontal normal stress are different, there exist initial shear. Because of this initial shear, apparent shear strength reduces resulting in smaller acceleration. This decrease of apparent shear strength is true in fresh ground, i.e., the ground that did not suffer earthquake in the past. If the soil was subjected to large earthquake in the past, however, the soil will behave as if it is in isotropic stress condition [Yoshida, 1996].

As the author pointed out [Yoshida, 1995], soft clay layer Ma13 that is located beneath the fill layer shows strong nonlinear behavior, which is also seen in Figure 7, which controls overall behavior of the ground above it. Since this layer is sufficiently old, it should behave like a soil with isotropic stress condition. This means that shear strength used in the analysis is underestimated because of fresh soil assumption. In order to confirm it, shear strength of this layer is increased keeping other condition unchanged, which result in Figure 8. The agreement becomes significantly improved. Especially the agreement at GL-16m is good. This again proves that the clay layer controls the behavior of ground above it. In the following analyses, however, shear strength is not adjusted partly because this does not affect the overall behavior and partly because this effect is common for both improved and unimproved sites.

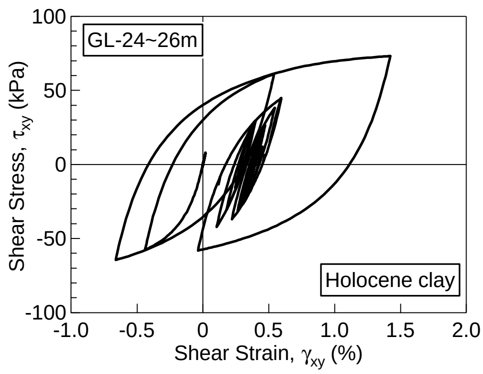


Figure 7: Stress-strain curve at Holocene clay layer

Excess porewater pressure ratios are shown in Figure 9, in which a result in one representative sublayer is shown in each layer. The same layer behaves nearly identical as can be seen in B2 up and B2 down layer in Figure 9(a).

Same with previous analysis, liquefaction occurs in the unimproved ground. Liquefaction also occurs in the deep layer in the improved ground, but it does not occur near the surface. The excess porewater pressure ratios in these layer are about 0.5, which is far from liquefaction.

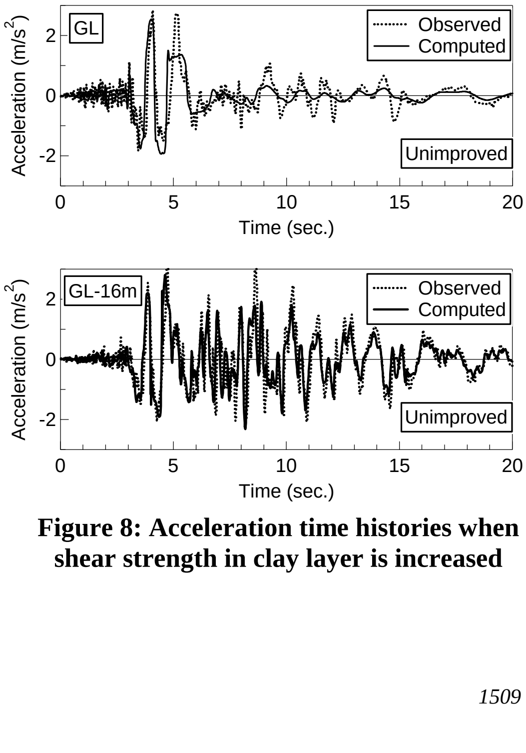
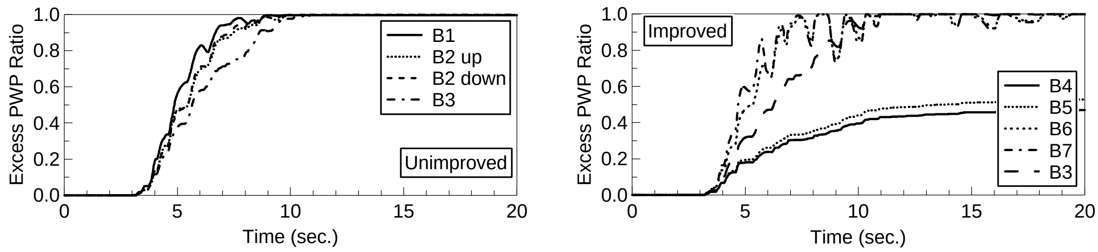


Figure 8: Acceleration time histories when shear strength in clay layer is increased

Figures 10 and 11 shows stress-strain curves and stress path of the liquefied layer in both unimproved and improved sites. Both layers liquefied because effective mean stress reaches nearly zero. However, as can be seen in the figures, the behavior is quite different. Hardening behavior is seen in the improved ground, which is caused by cyclic mobility behavior. On the other hand, soil directly goes to liquefaction in the unimproved ground. Therefore, although we call both phenomena as liquefaction, the behavior of soil



(a) Unimproved site (b) Improved site

Figure 9: Excess porewater pressure ratio time history

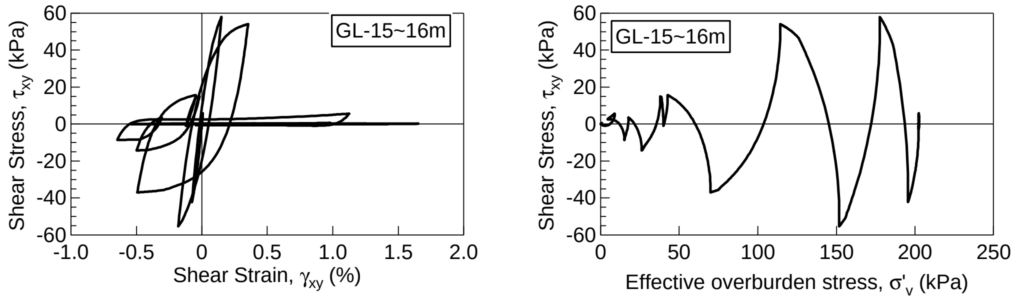


Figure 10 Stress-strain curve and stress path in unimproved site

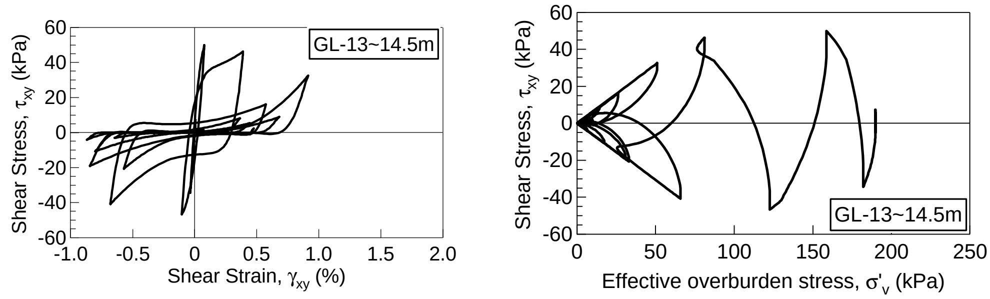


Figure 11 Stress-strain curve and stress path in improved site

cannot be discussed by the same word 'liquefaction'.

Acceleration time histories in the improved and unimproved sites are compared in Figure 12. In spite of the clear difference on the occurrence of liquefaction, accelerations at the ground surface are quite similar to each other. This can be explained by considering the following reasons.

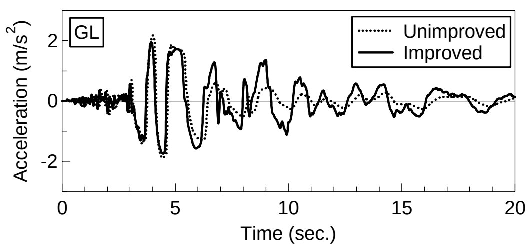


Figure 12: Comparison of acceleration time history between improved and unimproved sites.

As discussed in the preceding, the behavior of soft clay layer beneath the fill affects the behavior in the surface layer. Because of the strong nonlinear behavior in this layer, high frequency component is lost in the waves incident to the fill layer. Therefore, peak acceleration is strongly controlled by the predominant wave component.

Second reason is the time when peak response occurs. As seen in Figures 6 and 12, peak acceleration occurs at about 4 seconds, i.e., at the time when first significant wave arrives. When looking at Figures 10 and 11, soils still can resists shear stress and there is no significant difference in excess porewater pressure generation in both soils. Figure 9 indicates that liquefaction occurs at about 7 seconds at both sites. When looking at Figures 6 and

12, predominant wave already finishes. In other words, soils have shear resistance during the peak response. This reduces the difference between the responses at both layers.

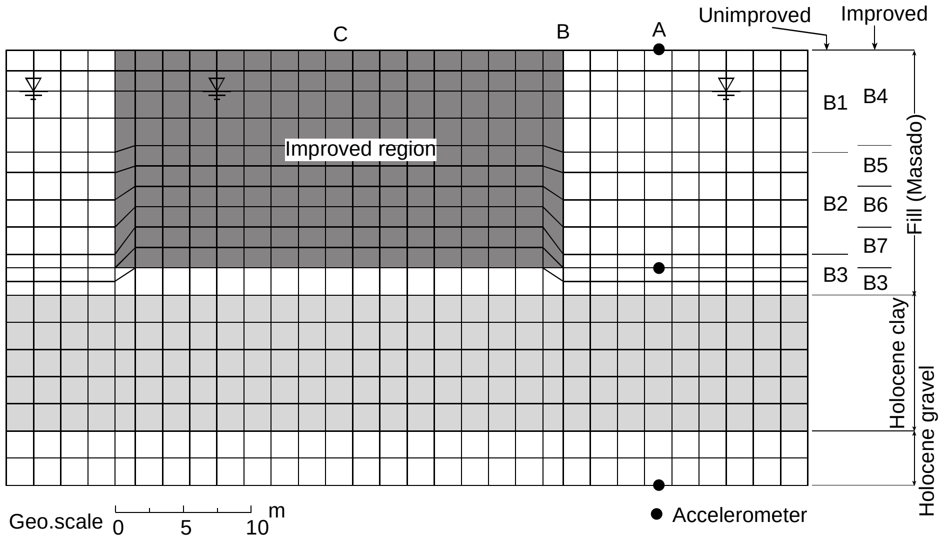


Figure 13: FEM mesh and models of soil layers

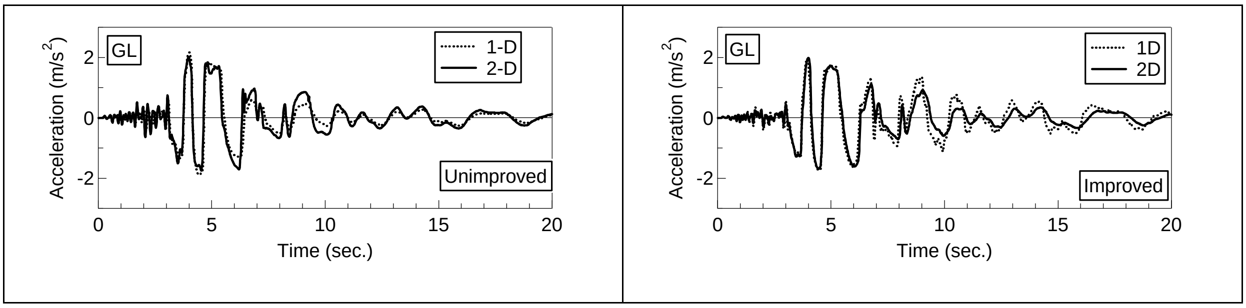


Figure 14: Comparison between 1-dimensional and 2-dimensional analyses

TWO DIMENSIONAL ANALYSIS AND DISCUSSION

EW section passing the strong motion observation station was chosen for 2-dimensional liquefaction analysis. Figure 13 shows FEM mesh. The improved region in the right in Figure 2 is modeled as it is. The area up to the unimproved area between two improved regions is modeled. Soil profiles in improved and unimproved sites are shown at the right side of the figure. The responses at Point A and B are used as representative response in the unimproved and improved grounds, respectively. Point B locates boundary between improved and unimproved ground.

Figure 14 compares accelerations at the ground surface by 1-dimensional and 2-dimensional analyses. There is no significant difference up to about 6 seconds, when peak response finishes. This can be recognized from the reason same with 1-dimensional analysis. After that, however, acceleration by 2-D analysis is larger than that by 1-D analysis in the unimproved ground whereas it is smaller than that by 1-D analysis in the improved ground. This indicates iterative behavior occurs between improved and unimproved regions.

Accelerations at points A, B and C are compared in Figure 15. All three accelerations are almost identical. Looking at Figure 12 where accelerations obtained by 1-dimensional analysis at the improved and unimproved

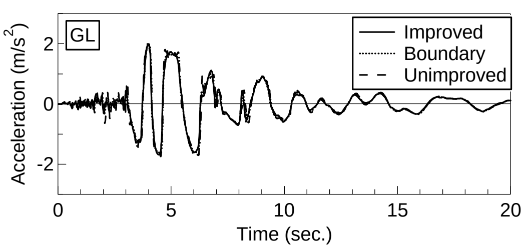


Figure 15: Comparison of acceleration at ground surface

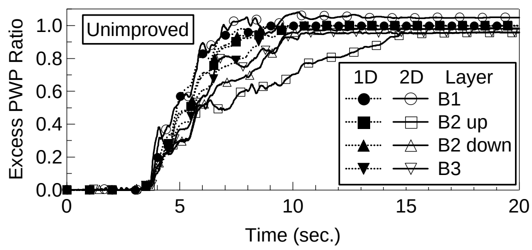


Figure 16: Comparison of excess porewater time history

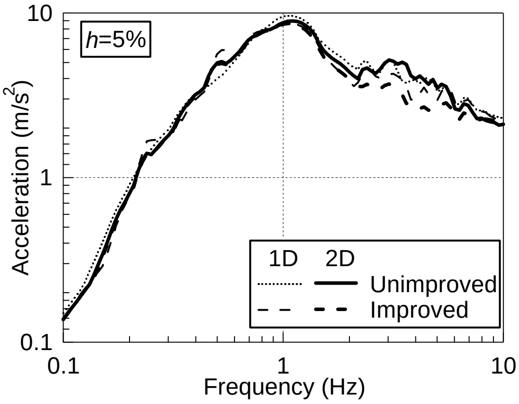


Figure 17: Response spectrum

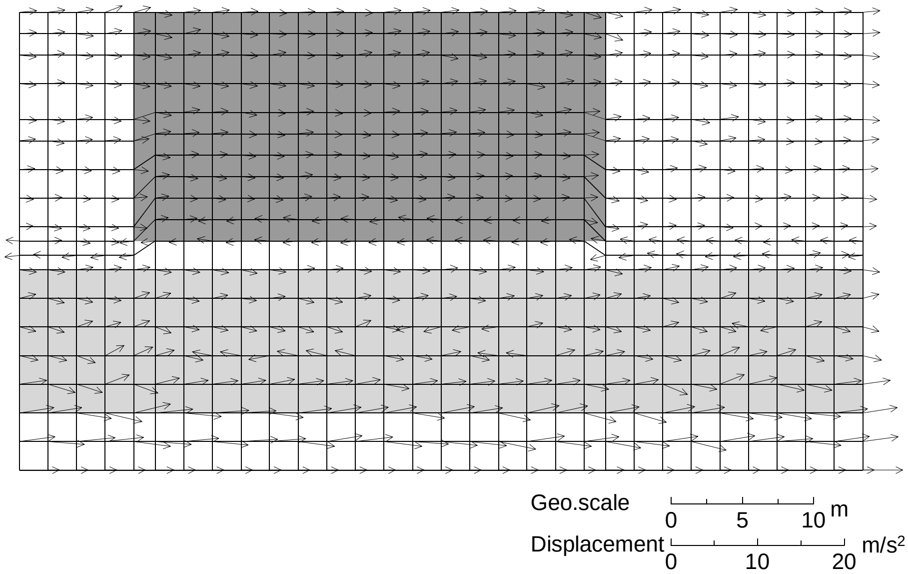


Figure 18: Distribution of peak acceleration. Direction of arrow indicates direction where peak response occurs.

sites are compared, although there was not significant difference until peak response finishes, behavior after 6 seconds differs fairly large. The improved site shows larger acceleration than the unimproved site because of the cyclic mobility behavior in the improved site as shown in Figure 11. Figure 14 indicates that dynamic interaction between the improved and unimproved sites works so that this difference reduces. As the result, the ground surface behaves identical as seen in Figure 15.

Effect of dynamic iteration can also be seen in the excess porewater pressure generation as shown in Figure 16 in which excess porewater pressure ratios by 1-dimensional and 2-dimensional analyses are compared. Since shear deformation in the unimproved region is constrained by the existence of improved region, excess porewater pressure generation in the unimproved layer is suppressed compared with one-dimensional analysis.

Figure 17 compares acceleration response spectrum under 5% damping. The responses are nearly the same up to 2 Hz, but differences are observed in high frequency region. It is noted that there seems no difference in accelerations in 2-dimensional analysis as seen in Figure 15, but, differences exists in high frequency region, resulting in the difference in acceleration response in high frequencies.

Figure 18 shows peak acceleration distribution in 2-dimensional analysis. Firstly, it is noted that peak acceleration deamplified in the Holocene clay layer because of strong nonlinear behavior. Looking at the surface layer, generally, the arrow directs horizontal. However, up-down component can be seen in the boundary between the improved and unimproved regions. This again indicates that iterative behavior occurs between improved and unimproved regions.

CONCLUDING REMARKS

The behavior of improved ground against liquefaction is investigated by the dynamic response analysis based on effective stress. Deep layer in the improved ground liquefies resulting in settlement of about 10 cm. It is noted, however, that the structure was not damaged although liquefaction occurred.

Effect of the existence of improved site that is located very close to the strong motion observation station at Port Island on the vertical array record is investigated by 2-dimensional effective stress analysis. Partly because peak accelerations occurs before soil liquefy and partly because nonlinear behavior of soft clay layer beneath the fill controls the behavior of surface layer, peak response was not affected. This also results in no effect on the acceleration response spectrum because the value is strongly controlled by peak input acceleration. After unimproved site liquefied, however, strong iterative behavior was observed between improved and unimproved sites, which works to make the acceleration response at the ground surface identical.

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