A STUDY ON DAMAGE TO STEEL PIPE PILE FOUNDATION ON RECLAIMED LAND DURING HYOGO-KEN-NANBU EARTHQUAKE

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SUMMARY

Damage investigation was conducted on steel pipe pile foundations damaged during the 1995 Hyogoken-Nambu earthquake. The investigation site was located about 100m from both north and west quay walls. According to the damage investigation, piles were deformed at two levels. One-dimensional effective stress analysis and soil-pile-structure interaction analysis by a simple spring-mass model considering liquefaction were performed to examine the behavior of the ground and structure during the earthquake. The analytical model was set in detail based on the results of soil investigations, laboratory test and design documents of the structure. According to the one-dimensional effective stress analysis, it is considered that reclaimed decomposed granite soil, "*Masado,*" below the ground water level liquefied. In soil-pile-structure interaction analysis, large bending moments over the full plastic moment appeared around G.L.-6m near the boundary of shear wave velocity in reclaimed layer, and around G.L.-15m, the lower boundary of *Masado* layer, during the main phases of the earthquake. The levels of large moment coincided with the levels, where the shear strains of the ground changed rapidly in regard to the vertical direction, obtained by one-dimensional effective stress analysis. And the levels agreed with those of pile deformation found in the damage investigation. It is therefore considered that the failure of piles occurred during the main phases of the motion. However, these analyses could not explain the residual displacements of piles, because large residual displacement of ground and piles did not appear in these analyses in which the layers were assumed to be horizontally stratified. It is considered that piles damaged by the liquefaction during the main phases of the earthquake, and additionally, deformed residually due to liquefaction-induced lateral flow of the ground toward quay walls.

INTRODUCTION

During the 1995 Hyogoken-Nambu earthquake, severe liquefaction occurred in wide areas of reclaimed lands, resulting in damage to many structures. A building structure on steel pipe pile foundations located in reclaimed land was also damaged in the disaster. The damage to the piles was investigated and subsequently analyzed based on the investigation results to find the causes of the damage. As a result, it was found highly probable that liquefaction and liquefaction-induced lateral flow of the ground caused failure of the piles [Kato et al., 1998 and Miwa et al., 1998].

In this study, one-dimensional effective stress analysis of ground and soil-pile-structure interaction analysis considering liquefaction were performed to examine the causes and the processes of the damage.

OUTLINE OF THE BUILDING AND GROUND

Figure 1 shows the location of the building to examine. The two-story building, approximately 100 m away from quay walls both in the north and in the west, was of steel-framed structure, excepting that the columns of the first

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OUTLINE OF THE BUILDING AND GROUND

Figure 1 shows the location of the building to examine. The two-story building, approximately 100 m away from quay walls both in the north and in the west, was of steel-framed structure, excepting that the columns of the first floor are made of reinforced concrete. It had a slender plan roughly stretching in the east-west directions, the long and short sides being 187 m (11 spans) and 18 m (1 span), respectively. The foundations consisted of independence footings, each supported by 4 or 6 steel pipe piles (diameter is 406.4 mm, thickness is 9.5 mm and length is 27.5m).

Soil investigations were conducted in 1968 before the earthquake, in 1996 and in 1997 after the earthquake. 15 investigation points in total, indicated in Figure 1. Figure 2 shows the soil profiles and standard penetration test results "SPT *N*-value" for 11 points along the crossbeam direction of the building. From the ground surface (K.P.+3.6 m) to the depth of K.P.-10 m, approximately 14 m in thickness is a reclaimed layer comprising *Masado*, decomposed granite soil. The SPT *N*-values are relatively high at 10 to 30 in the range of K.P.+3.6 to ±0 m, which is above the ground water level (K.P.-0.4 m) in the subsurface range. There are also high at 20 to 40

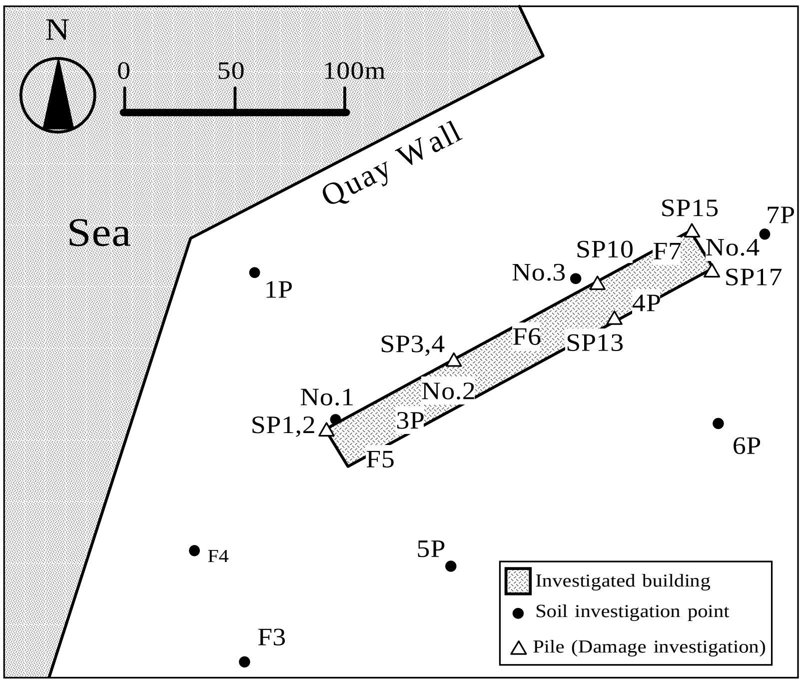


Figure 1: Location of the investigated building and soil investigation points

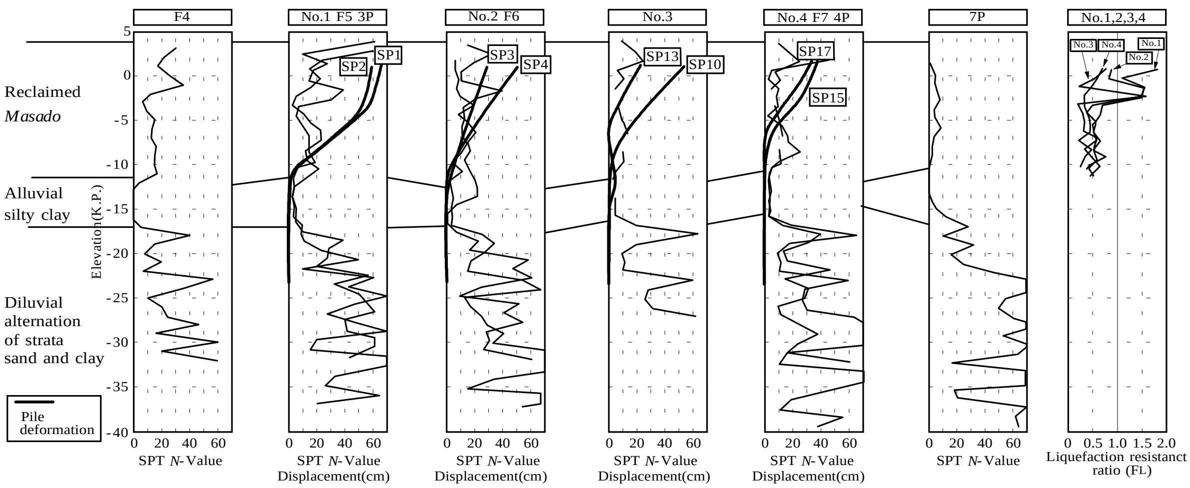


Figure 2: Soil profiles (East-West section), pile deformations and results of the liquefaction evaluation

at a level of K.P. –2 to -3 m, whereas under K.P. –2 to -3 m, in 8m thick *Masado* layer, there are low SPT *N*-values between 0 and 15. All the soil profiles show the bottom of an alluvial silty clay layer at K.P.-15 to -17 m and the bottom of the *Masado* layer at K.P. -10 to -11 m, considering that the ground around the investigation site is horizontally stratified.

An array observation of earthquake motions has been continued at Higashi Kobe Bridge near the investigation site, providing strong motion records of Hyogoken-Nambu earthquake at G.L.-1.5 m and G.L.-34 m [Hagiwara et al., 1995]. A simplified liquefaction evaluation was carried out for investigation site of No.1 to No.4 by the method specified in the Recommendations for Design of Building Foundations [Architectural Institute of Japan, 1988], assuming the maximum acceleration at the ground surface to be 400 cm/s^2 based on the observation record at G.L.-1.5 m. The results are shown in Figure 2. The liquefaction resistant ratio "F_L" is less than 1 in most of the *Masado* layer excepting the upper part, considering a high possibility of liquefaction in this layer.

The deformation of piles found in the damage investigation is superimposed on Figure 2. The depths of significant deformation correspond to the bottom of the *Masado* layer, which presumably liquefied, and near K.P.-2 m, at which the SPT *N*-value was low.

ANALYSIS OF THE GROUND BEHAVIOR DURING THE EARTHQUAKE

Modeling of ground

The ground around the investigation site being nearly horizontally stratified, it is considered that the behavior of the ground can be evaluated using a one-dimensional model, if the effects of lateral flow near the quay walls are excluded. A one-dimensional effective stress analysis was therefore conducted to evaluate the ground behavior during the earthquake.

"*FLIP*" was used as the analysis code [Iai et al., 1990]. Hysteretic hyperbolic model were applied to the nonlinear relationship between shear stress and shear strain. Figure 3 shows the analytical model of the ground, in which the ground down to G.L.-33.7 m is modeled in detail based on the results of the soil investigations and laboratory tests. The *Masado* layer between G.L.-3.8 m and G.L.-14.75 m was assumed to be a liquefiable soil layer.

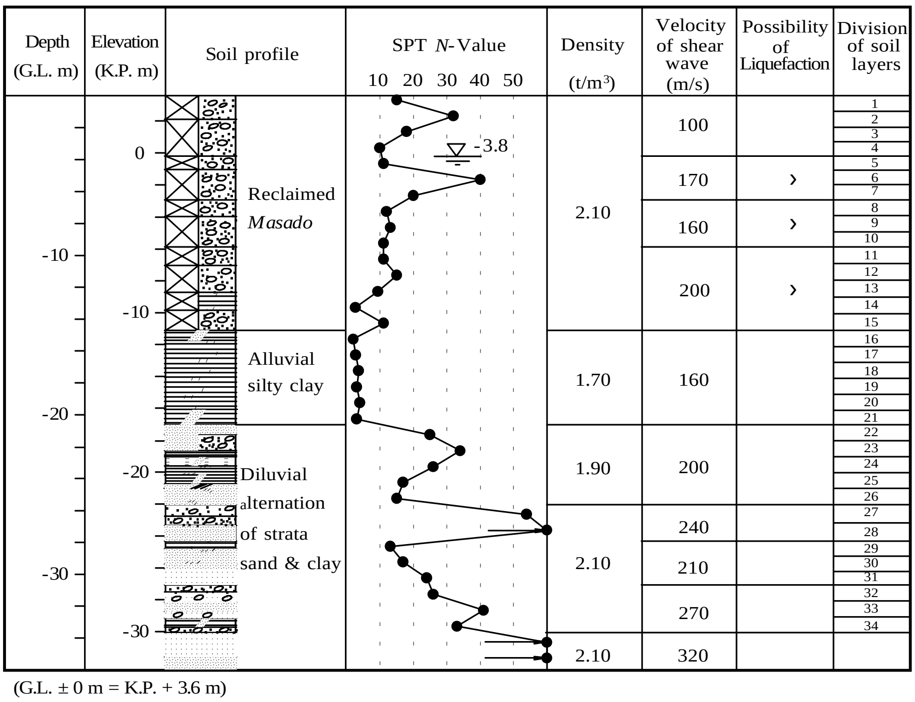


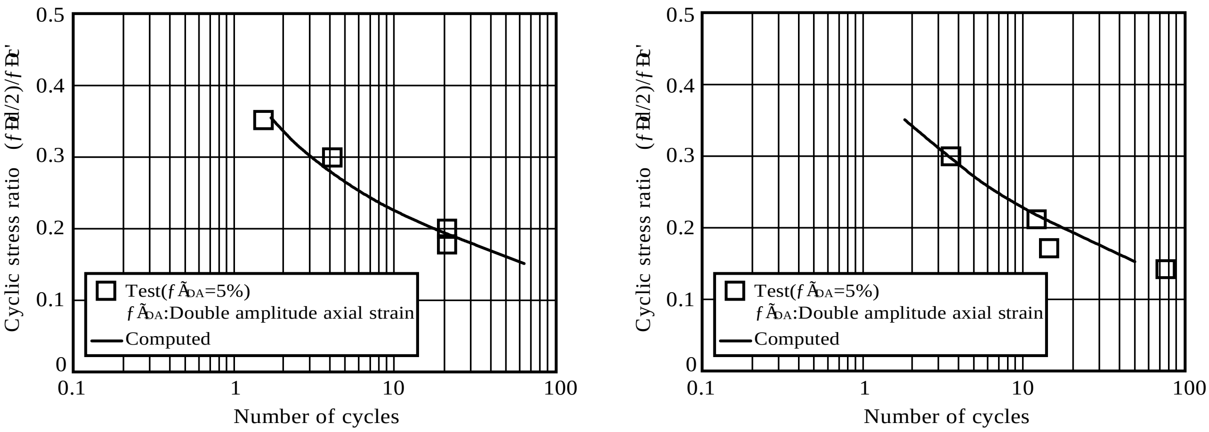
Figure 3: Analytical model of ground

There are two kinds of parameters to use for analysis. One is the parameter of nonlinear characteristics that defines the relationship between shear stress and shear strain. The other is parameter of liquefaction strength affecting the rise of the excess pore water pressure. The parameters of nonlinear characteristics were determined so that the hyperbolic models may agree with the nonlinear characteristics of soils obtained by laboratory tests.

The liquefaction strength of the soil was obtained by cyclic triaxial tests on undisturbed samples taken by triple-tube sampling from between G.L.-8.0 m and G.L.-8.9 m (sample A) and between G.L.-10.5 m and G.L.-12.2 m (sample B). Figure 4 shows the liquefaction strength. The cyclic stress ratios, required to cause 5% double amplitude axial strain in 20 cycles, obtained by the tests are about 0.2. The cyclic stress ratios are similar to past test results of undisturbed samples of *Masado* using triple-tube sampling taken from nearby reclaimed lands [Mizutori et al., 1998].

The liquefaction strengths of sample A and B were applied to the *Masado* layer at G.L.-3.8 to -9.5 m and G.L.-9.5 to -14.8 m, respectively. The parameters of liquefaction strength of soil were determined to fit the liquefaction strength obtained by the elemental simulation analysis of simple shear test using "*FLIP*" to that of soil obtained by laboratory tests. The liquefaction resistance curve calculated from simulation analysis is superimposed on Figure 4. The liquefaction resistance curve well fitted to the liquefaction strength obtained by the cyclic triaxial tests, considering the validity of the parameter setting.

The fines content of *Masado* obtained by the laboratory test was between 5 % and 15 %, with the SPT *N*-value ranging from 5 to 10. The liquefaction strength estimated from the SPT *N*-value ranges from 0.1 to 0.18, using the simplified liquefaction evaluation method based on the Recommendations for Design of Building Foundations [Architectural Institute of Japan, 1988]. These values nearly equal to the liquefaction strength obtained by the laboratory cyclic triaxial test, in consideration of the change of confining pressures in regard to the vertical direction. Accordingly, the liquefaction strength estimated from the SPT *N*-value was found adequate.



(a)G.L.-8.0 ` G.L.-8.9m (b)G.L.-10.5m ` G.L.-12.2m

Figure 4: Cyclic triaxial test results of *Masado* and computed liquefaction resistance curve

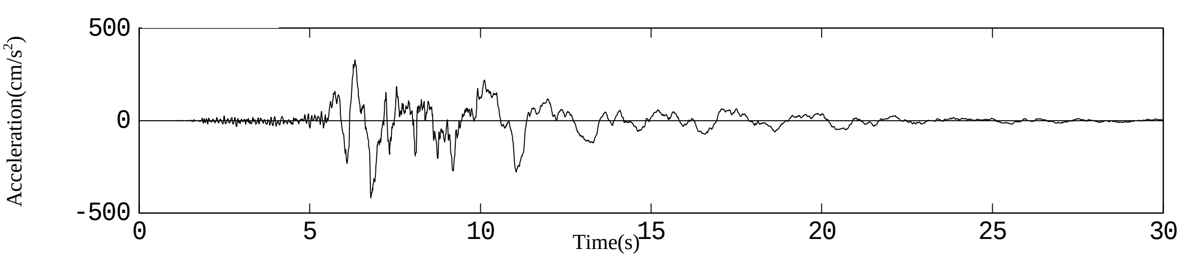


Figure 5: Input motion for response analysis

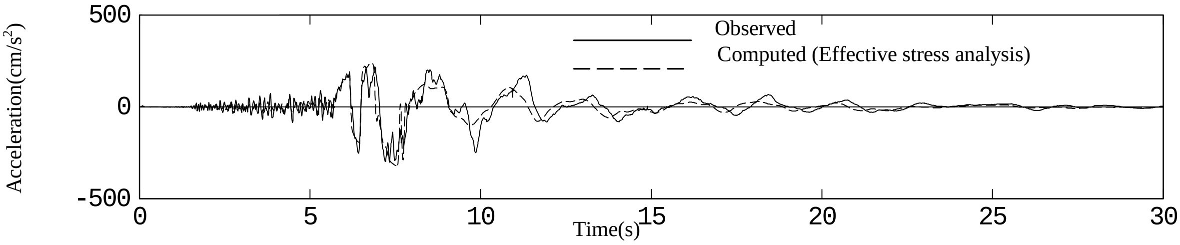


Figure 6: Comparison between observed and computed acceleration time history at the surface

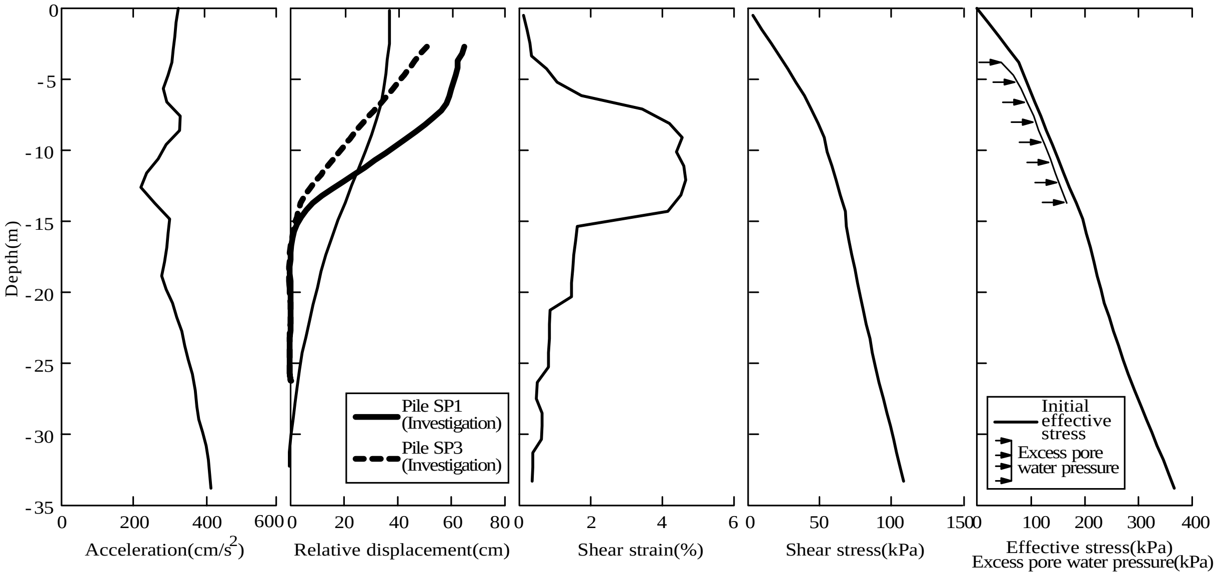


Figure 7: Maximum response values of ground by one-dimensional effective stress analysis and

The observed record at G.L.-34 m in the Higashi Kobe Bridge Station was used as the input motion. Strong motion records were rotated for analysis to N 334 E direction that was the span direction of the building. The wave was inputted as the incident/reflection wave (E+F input) to G.L.-33.7 m of analytical model. The sand layer under the depth of G.L.-33.7 m was regarded as a base, shear wave velocity of the layer being larger than 300m/s. The acceleration time history of the input motion is shown Figure 5.

Ground behavior during the earthquake

Figure 6 shows the time history of acceleration at the surface calculated from the effective stress analysis. The waveform observed at G.L.-1.5 m in the Higashi Kobe Bridge Station, rotated similarly to the input motion, is also shown for comparison. The computed motion reproduced the features of the observed motion, such as the predominant period was extended and amplitude was reduced after the main phases.

Figure 7 shows the vertical distribution of the computed maximum acceleration, relative displacement, shear strain, shear stress and excess pore water pressure. The excess pore water pressure in the *Masado* layer, below the ground water level, is increased to more than 90 % of the initial effective stress of the ground. Also, the shear strain level exceeded 2% excepting the range from the ground water level to G.L.-6 m, in which the SPT *N*-values are relatively high. Accordingly, it is considered that the *Masado* layer from G.L.-6 to 15m was liquefied during earthquake.

Deformation of piles found by damage investigation, pile SP1 and SP3, are superimposed in the Figure 7. Pile SP1 was deformed at around G.L.-6 m and G.L.-15 m, whereas pile SP3 was deformed at around G.L.-15 m. The computed shear strain of ground was suddenly changes at G.L.-6 m and G.L.-15 m, suggesting a possibility that the piles were damaged at the depth of strain changing when liquefaction occurred.

ANALYSIS OF THE BEHAVIOR OF THE PILES DURING THE EARTHQUAKE

Modeling of structure

A seismic response analysis of soil-pile-structure interaction was conducted using a multi-lumped mass model considering liquefaction proposed by Mori et al. [Mori et al., 1992] to evaluate the behavior of the piles and building during the earthquake as well as the causes and the processes of damage. One frame of span direction was modeled for the analysis. The analytical model, that is an improved model of so-called modified Penzien type model, is shown in Figure 8.

This model consists of a structure-pile system, near field system, and free field system. The piles were assumed to move identically with the near field in horizontal direction. These systems and the free field system were connected by interaction springs at the corresponding masses. The free field system is a lumped masses and shear springs model. The hysteretic hyperbolic model was applied for nonlinear relation between shear stress and shear strain of the interaction spring. Also, nonlinearity of stiffness and strength of spring depending on the change of effective confining pressure due to the building up of the excess pore water pressure by liquefaction was considered. The parameter of spring, maximum strength and maximum stiffness, were adjusted to be proportional to the one half power of the effective confining pressure. The response of the free field was obtained by the one-dimensional effective stress analysis, "*FLIP*". The structure was modeled by a one mass spring system, and the piles and structure were modeled by linear beam elements in these analysis.

Pile behavior during the earthquake

Vertical distribution of the maximum response value of pile computed by soil-pile-structure interaction analysis, acceleration, relative velocity, relative displacement, bending moment and shear force, are shown in Figure 9. The bending moment and shear force values represent the collective values for 12 piles of a single frame. Maximum ground response values are superimposed in the Figure 9. Behavior of piles agrees with behavior of the ground excepting the surface layer. It is considered that behavior of piles were mainly controlled by the ground deformation in this case. The difference between the behavior of the ground and those of piles at the surface layer suggests the influence of the inertial force of the building.

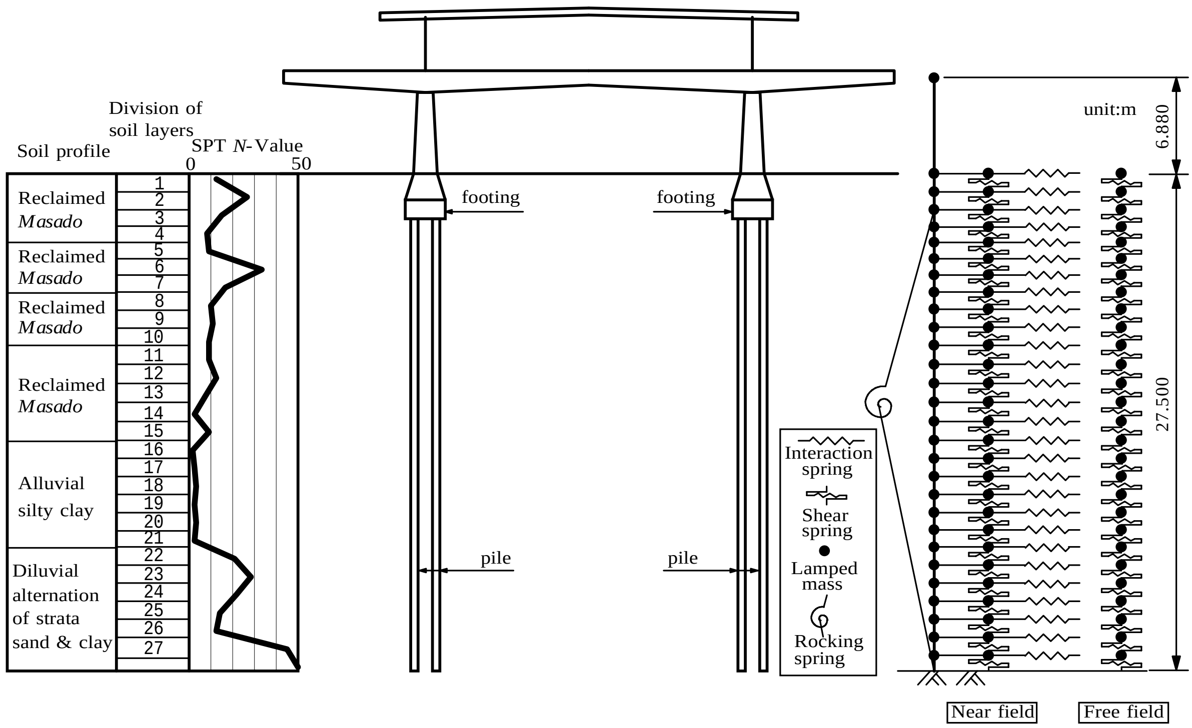


Figure 8: Analytical model of soil-pile-structure interaction

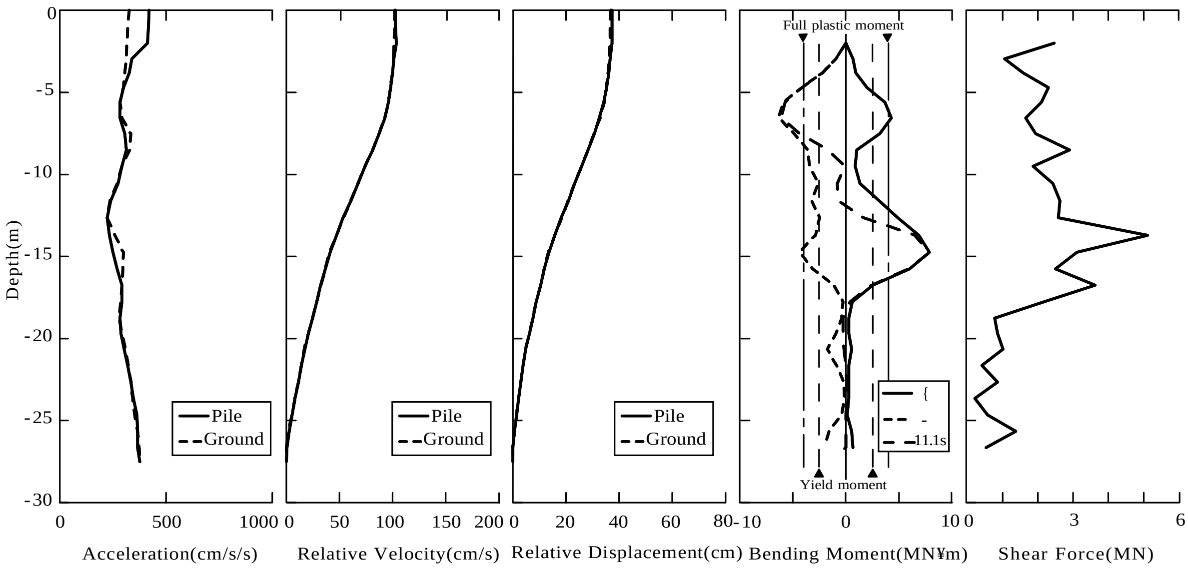


Figure 9: Maximum response values of pile and ground computed by soil-pile-structure interaction analysis

The distribution of bending moment at 11.1 seconds, at the time the maximum moment appeared is also shown in the figure for the bending moment. The maximum bending moment at G.L.-4 to -8m and G.L.-15m appeared simultaneously and exceeded the full plastic moment during the main phases. These depths agreed with those piles deformed. The direction of the bending moment were opposite at the upper and lower parts, and these directions agree with those of the moment that may caused the deformation of the piles found in the damage investigation, respectively. It is considered that the failure of piles at the positions of residual deformation occurred during main phases of the earthquake and developed thereafter. The maximum bending stress and shear stress of the R.C. columns were more than three times the allowable stresses. It is therefore considered that the columns also failed in the main phases.

However, these analyses could not explain the residual displacements of piles, because large residual displacement of ground and piles did not appear in these analyses in which the layers were assumed to be horizontally stratified. According to the aerial photo survey, residual horizontal displacements of transverse direction at quay walls near the site were 2.6m to 4.6m, those damage were considered to be caused by the liquefaction-induced lateral flow of the ground. And the piles deformed to quay wall found in the damage investigation suggesting the possibility of the influence lateral flow. Lateral displacement estimated using simplified method agreed with those results [Miwa et al., 1998]. As a result, it is considered that piles damaged by the liquefaction, and additionally, deformed residually due to liquefaction-induced lateral flow of the ground toward quay walls in spite of distance of more than 100m from the quay walls.

CONCLUSIONS

A one-dimensional effective stress analyses of the ground and a soil-pile-structure interaction analysis incorporating the effects of liquefaction were conducted in regard to structure on steel pipe pile foundations damaged during the 1995 Hyogoken-Nambu earthquake in reclaimed land. The results were compared with the damage investigation to examine the causes and processes of the damage. As a result, the following were found:
(1) The soil layers around investigation site could be regarded almost horizontally stratified.
(2) The liquefaction strength of Masado layer, obtained by the cyclic triaxial tests on undisturbed samples taken by triple-tube sampling, were nearly equal to those by past tests nearby reclaimed lands. These values agree with the strength estimated from SPT *N*-value.
(3) The dynamic response of liquefied ground was expressed satisfactorily by the one–dimensional effective stress analysis using the model based on soil investigation and laboratory tests. The depths where the strains of the ground changed rapidly in regard to the vertical direction agreed with those of pile deformation found in the damage investigation.

(4) By the analysis of the soil-pile-structure interaction, large bending moments beyond the full plastic moment appeared around G.L.-6m near the boundary of shear wave velocity in reclaimed layer, and around G.L.-15m, the lower boundary of *Mmasado* layer, during the main phases of the earthquake. The levels and direction of large moment coincided with those of pile deformation found by the damage investigation. It is therefore considered that the failure of piles occurred during the main phases of the earthquake.
(5) The analysis of the soil-pile-structure interaction assuming a stratified ground did not lead to significant residual displacements. The residual displacements of the piles and ground could not be fully explained by the response of horizontally stratified ground with liquefaction.
(6) From these analyses and investigations, it is considered that piles damaged by the liquefaction during the main phases of the earthquake, and additionally, deformed residually due to liquefaction-induced lateral flow of the ground toward quay walls in spite of distance of more than 100m from the quay walls.

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