# DAMAGE ANALYSIS OF A PILE FOUNDATION AND SEISMIC ISSUES REVEALED BY THE 1995 GREAT HANSHIN EARTHQUAKE

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**ABSTRACT**

This paper presents the damage to the pile foundation of a bridge on the Hanshin Expressway caused by the 1995 Great Hanshin Earthquake. The cause of the damage to the pile foundation was studied by means of the three-dimensional finite element method taking into account soil liquefaction and the lateral spreading of the liquefied soil. The pier of the bridge moved about one meter laterally toward the river; the revetment moved about three meters laterally in the same direction and subsided by more than one meter. The analysis revealed that the damage was mainly caused by ground motion (kinematic interaction) during the earthquake, and that the damage was magnified by the lateral spreading of the liquefied soil which occurred in succession after the main shock, causing large residual deformation to the piles. Remedial measures involving the adoption of sand-compaction piles or steel-pipe-sheet piles were investigated in order to assess analytically the level to which the damage could be mitigated. The steel-pipe-sheet piles constructed between the foundation of the pier and the quay wall can withstand strong motion, soil liquefaction, and the lateral spreading it induces. Apropos the 1995 event, an urgent need to examine the cause of the damage and to study remedial measures to mitigate similar damage in Japan became apparent because as with this bridge, many key structures have been constructed on reclaimed land. Seismic issues related to pile foundations revealed by the 1995 earthquake are demonstrated from the view of the damage to pile foundations.

**KEYWORDS**

Pile foundation, the 1995 Great Hanshin Earthquake, soil liquefaction, lateral spreading induced by soil liquefaction, remedial measure, 3D-FEM, soil-structure interaction, highway bridge, damage analysis

## INTRODUCTION

Serious damage to pile foundations was exposed after the January 17, 1995 Great Hanshin Earthquake ($M_{JMA}$=7.2 on the Japan Meteorological Agency scale magnitude, Focal depth; D=14 km). To investigate the damage to the pile foundations, considerable labor and funds were required because the structure was underground and consequently the reports detailing the damage to the pile foundations were published a few months after the 1995 event. The importance of seismic issues relating to pile foundations has been recognized anew as the truth of the damage sustained during the 1995 earthquake was made public, and accordingly, research on these issues has been vigorously pursued since the Great Hanshin Earthquake.

The damage to structures in Kobe, which was substantially heavier than was anticipated, has brought even the basis of earthquake engineering into question. As for pile foundations, which is better for the seismic performance; that a number of piles with smaller diameters are used or a few larger diameter piles is adopted? Could the damage have been mitigated or prevented if more piles had been installed or piles with larger diameter had been used, or if soil improvement had been carried out to prevent soil liquefaction? Do all of the existing pile foundations designed on the basis of old design codes need strengthening or repairing? If the renovations are not carried out, what types of problems will arise in the event of a major earthquake?

There are very few studies that can adequately provide answers to the questions mentioned above, despite the fact that they are very straightforward questions for engineers dealing with daily design operations and the construction of pile foundations. The research on pile foundations has a long history and many researchers have been active in this field. However, a number of essential issues related to pile foundations were brought to light by the damage caused by the 1995 earthquake, issues for researches and engineers working in this field for which no suitable solutions have been realized and this generated surprise.

During recent decades, pile foundations have been utilized in the majority of the foundations of structures constructed on soft soil deposits, for economic reasons and in terms of the efficiency of execution. Viewed from the perspective of the existing state of construction, there is no room for delay in resolving these issues. However, the progress of research on these issues is very slow and has not been appreciable, and the researchers and engineers in this field are now faced with a major dilemma. In order to prepare for the next major earthquake, fast and dramatic progress in this research field is essential and can be achieved by integrating the knowledge on seismic engineering.

This paper presents the condition of the damage to the pile foundation of a bridge caused by the 1995 earthquake, and examines the cause by means of the three-dimensional finite element method taking into account soil liquefaction and the lateral spreading of the liquefied soil. Remedial measures involving the adoption of sand-compaction-piles or steel-pipe-sheet-piles are investigated in order to assess analytically the level to which the damage can be mitigated. Furthermore, the seismic issues related to the design and analysis of pile foundations are summarized from the perspective of the damage to pile foundations caused by the 1995 earthquake.

## ANALYSIS OF THE DAMAGE TO THE PILE FOUNDATION OF A HIGHWAY BRIDGE CAUSED BY SOIL LIQUEFACTION AND ITS LATERAL SPREADING

## OUTLINE OF THE BRIDGE AND THE DAMAGE

The damaged bridge shown in Photograph 1 has three spans of Gerber-steel-box girders 310 meters in length which cross a river running between the artificial islands of Nishinomiyahama and Minami-Ashiyahama. The bridge was completed in 1993, two years before the 1995 earthquake. The rigid-framed steel pier (pier No. 134 shown in Photo 1) on Minami-Ashiyahama Island is supported by 56 piles (4 rows x 14 columns) as shown in Figure 1, which are cast-in-place concrete piles, 34 meters in length and 1.5 meters in diameter. The soil deposit consists of a reclaimed soil layer (GL 0 ~ –14 m) mainly composed of gravel, an alluvial clay soil layer (GL –14 ~ –20 m), an alluvial sandy soil layer (GL –20 ~ –28 m), and a diluvial sandy soil layer (under GL –28 m). The soil profile of the ground can be seen in Figures 3 and 5 shown later. The ends of the piles are embedded in the diluvial sandy soil layer.

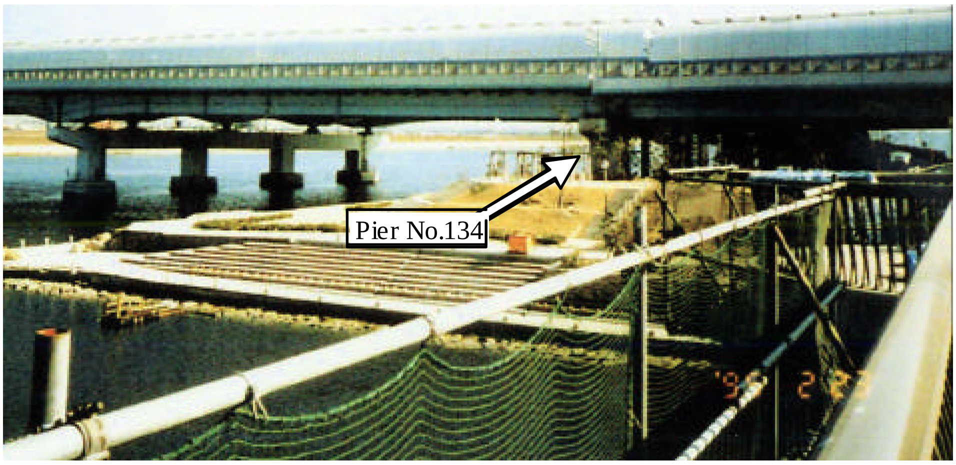



**Photo 1. The damaged bridge on Hanshin Expressway Route No. 5**
**(The photograph was taken from the north)**

Figure 2 illustrates the residual deformations of the pier and the revetment caused by the 1995 earthquake. The pier moved about one meter laterally toward the river. The revetment moved about three meters laterally in the same direction and subsided by more than one meter. Photograph 2, taken two weeks after the earthquake, shows the ground failure around the pier. The occurrence of soil liquefaction around the bridge was identified by evidence of excessive boiled sand flow across the entire surface of the surrounding ground. Severe damage to the shoe on the pier on Minami-Ashiyahama Island was found, however there was no damage to the pier itself. The pier located on Nishinomiyahama Island moved 0.6 meters laterally toward the river.

## DAMAGE TO THE PILES

Three investigation methods were adopted to survey the damage to the piles of the pier, namely direct inspection by excavation around the pile-heads, integrity sonic test utilizing elastic waves, and indirect inspection utilizing a borehole camera. Figure 3 shows the result of the investigation using the borehole camera. Inspection of the first borehole (borehole A) was halted at the depth of GL –21 m due to contact with a reinforcing bar. The second borehole (borehole B) was located at a distance of 40 cm from borehole A. The condition of the cracks in boreholes A and B was different. Many cracks appeared between the pile-head and

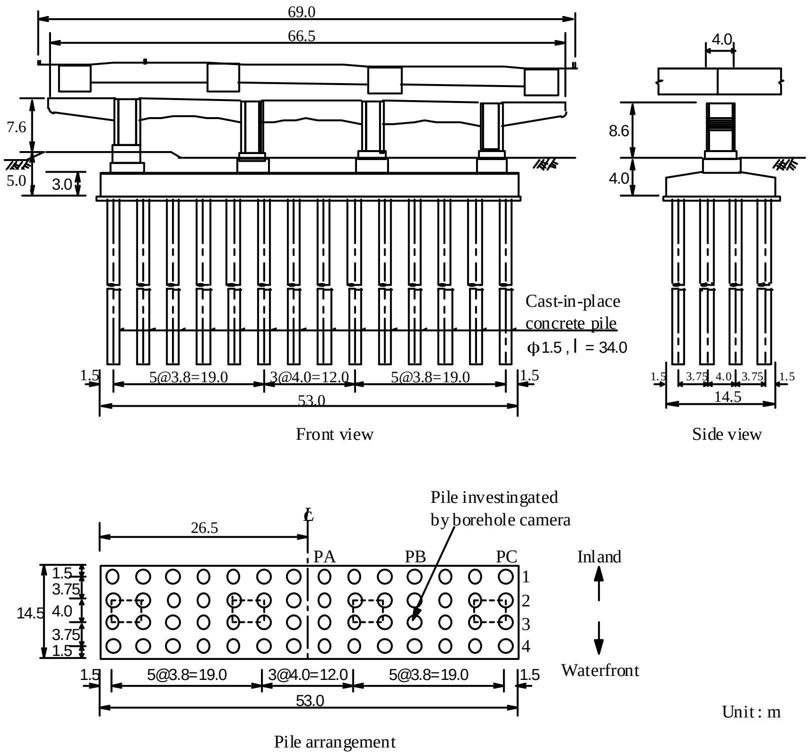



**Figure 1. Plan views of the pier and piles of the damaged bridge**

**(The cross section is viewed from the south)**

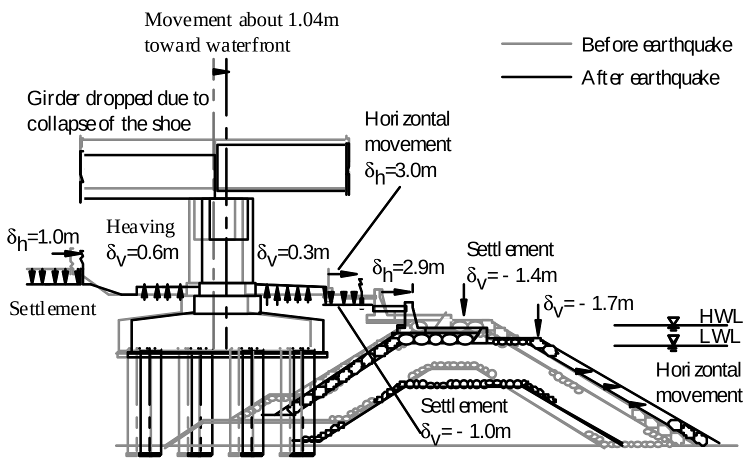



**Figure 2. Residual horizontal displacements and settlements of the pier and the revetment**

GL –12 m in borehole A. On the other hand, cracks at the depths of GL –10 m, –13 ~ –16 m, –20 ~ –23 m, and –35 m were observed in borehole B. The difference in the occurrence of the cracks between boreholes A and B represents the fact that the pile was remarkably deformed in one direction. The deformation of the pile shown in Figure 3 was obtained by evaluating the conditions of the opening or closing of the cracks. The field investigation identified one meter of lateral displacement toward the river at the pile-head.

**Photo 2. The damage to the ground in front of the pier No.134**
**(The photograph was taken from the north side of the pier)**

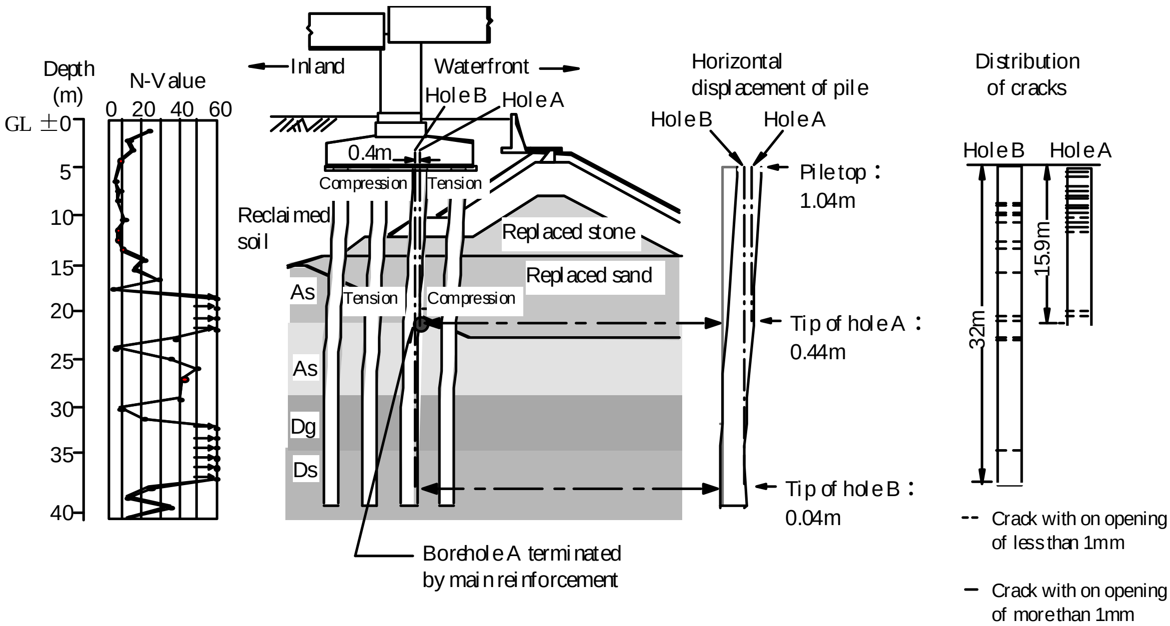



**Figure 3. Cracks and horizontal displacements in the pile observed using the borehole camera**
**(The cross section is viewed from the south)**

## ANALYTICAL PROCEDURES

In this study, separate evaluations of the dynamic response affected by soil liquefaction due to the shaking during the earthquake and the quasi-static response of lateral spreading of the liquefied soil were made. The procedure proposed by Ohtsuki et al. [Ohtsuki et al., 1998] was used in the soil liquefaction analysis. The modified Ramberg-Osgood and the bowl models [Fukutake and Matsuoka, 1989] were adopted respectively for the stress-strain and strain-dilatancy relationships in the procedure. Initial stresses in the ground are off balance because the shear modulus of the liquefied soil deposits, which have a high potential to be affected by excess pore water pressure, decrease remarkably and its value becomes very small. Thus, the liquefied soil deposits flow laterally along sloped soil layers due to the unbalanced stress. Lateral spreading resulting from soil liquefaction occurs continuously not only during the main shock but also for ten minutes after the shaking. The phenomenon of the lateral spreading of ground can be adequately simulated using static analysis due to the force of gravity, under the condition that the stiffness of the liquefied soil deposits decreases according to the accumulation of excess pore water pressure; this data is obtained from the analysis of soil liquefaction.

**ANALYTICAL MODEL AND INPUT MOTION**

Figure 4 shows the analytical model (a half model is used since the model is a plane symmetry model) of the pier, pile foundation, and ground for the three-dimensional finite element analysis. The numbers of nodes and elements in the half model are 10,539 and 9,557, respectively. The pier was modeled using a linear beam. The Takeda model [Takeda et al., 1970] was used to analyze the nonlinear behavior of the piles. Figure 5 shows the configuration of the soil layers of the model. The parameters for the nonlinearity and undrained shear strength of the soils were determined based on the data obtained from the laboratory tests of the sampled soils in situ.

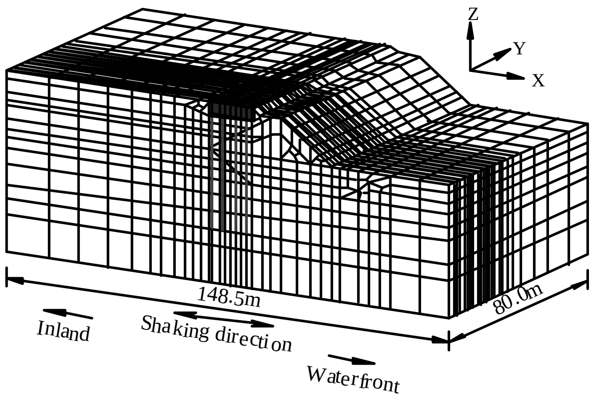



**Figure 4. The three-dimensional analytical model for the soil-piles-pier system (The front side is the symmetrical plane)**

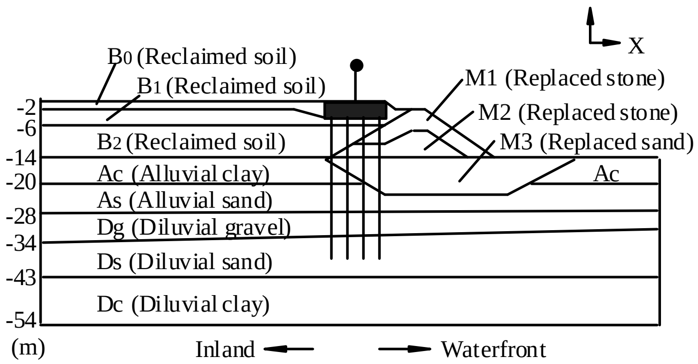



**Figure 5. The soil profile of the analytical model**

The input motion shown in Figure 6 was obtained from the calculation using two horizontal components recorded at GL –83 m in Port Island, which were transferred to the longitudinal component of the bridge axis. The observed point of the records is approximately 12 km from the pier. The maximum acceleration value of the input motion is 347 gal. The predominant periods of the free field of the ground and the pier when the bottom of the column of the pier is fixed to a rigid body are 0.93 and 0.14 seconds, respectively.

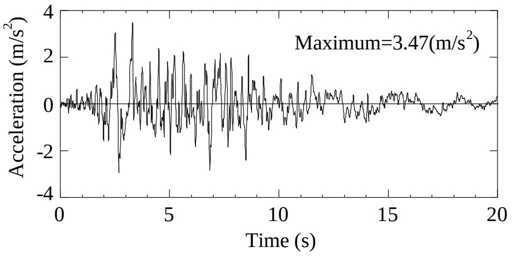



**Figure 6. Input motion (The records of two components observed at Port Island at a depth of 83m are transferred to the longitudinal and the orthogonal components of the bridge axis)**

**SOIL LIQUEFACTION ANALYSIS**

Figure 7 represents the maximum value of the excess pore water pressure ratio (excess pore water pressure / initial mean effective stress) obtained from the soil liquefaction analysis. It was possible to detect that the entire region of soil layers B2, As, M3, and a part of soil layer B1 had been liquefied since the excess pore water pressure ratio exceeded 0.95. Figure 8 shows the time histories of the excess pore water pressure ratio of soil layers B2 (GL –8.5 ~ –10.5 m) and As (GL –20 ~ –24 m), which are the underlying soil layers of the footing, and the time histories of the acceleration and displacement at the base of the pier. The peak of the acceleration at the base of the pier was appeared three or four seconds after the occurrence of the earthquake, according to the peak of the input motion. Soil layers As and B2 liquefied four and ten seconds respectively after the occurrence of the earthquake. The predominant period of acceleration increased after the peak, which was caused by the liquefaction of soil layer As after four seconds. The predominant period of the displacement also became longer after four seconds.

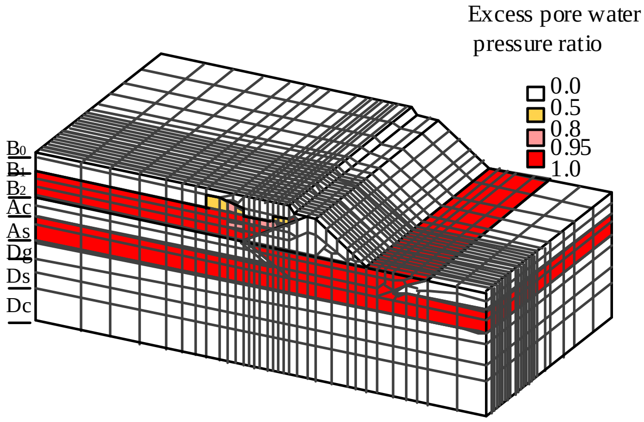
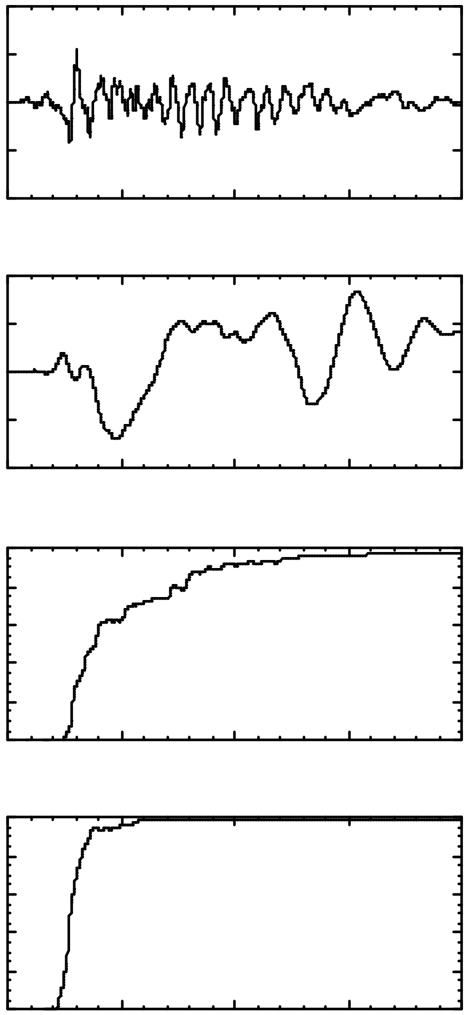



**Figure 7. Distribution of the excess pore water pressure ratio obtained from the soil liquefaction analysis**



The time histories of the bending moments at the pile-head of three piles (PA3, PB3, and PC3) are shown in Figure 9. PB3 corresponds to the pile investigated using the borehole camera. My and Mu in the figure represent the yield moment of the reinforcing bar and the ultimate bending moment of the pile, respectively.
There are no major differences between the bending moments of the three piles. The peaks of the bending moments close to the value of My appeared at 3 ~ 4 seconds, 7 ~ 9 seconds, and 15 seconds, respectively.
Figure 10 illustrates the distribution of the lateral displacements and the maximum bending moments of the three piles along their depths. The piles were bent largely in the liquefied soil layer As. The bending moments became significantly large at the upper and lower boundaries (GL –20 m and –27.5 m) of the soil layer As, since the soil stiffness changes enormously at these boundaries. The maximum bending moments which reached the yield value, My, of the pile obtained from the soil liquefaction analysis, corresponded to the positions of the observed cracks on the piles at the pile-head, GL –17 m, and GL –20 m.

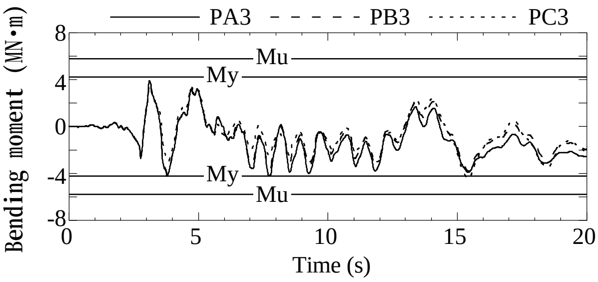
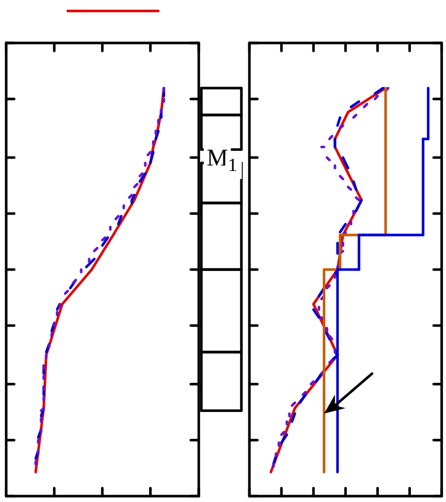



**Figure 9. Time history of the bending moments at the pile-head of the piles (PA3, PB3 and PC3)**




**THE CAUSE OF THE DAMAGE TO THE PILES**

To investigate the cause of the damage to the piles it is crucial to establish a rigorous seismic design method. It is a well known fact that the inertial force of superstructure and ground motion, namely inertial and kinematic interactions, dominate the seismic response of piles, however the extent to which each of the forces respectively affects the response of piles remains to be elucidated. We examined the quantitative identification of the cause of the damage due to the inertial force of the superstructure and ground motion.

To identify the effect of ground motion on pile behavior, we utilized soil liquefaction analysis on a model with the superstructure removed from the original model shown in Figure 4 and with a massless footing. The use of nonlinear analysis means that this is not a precise study, however the differences between the results in

Figures 9 and 10, and the results of this analysis, approximately represent the behavior caused exclusively by the inertial force of the superstructure. Figure 11 shows the results of the time histories of the bending moments at the pile-head. The solid and dashed lines represent the bending moments obtained from the models with the superstructure and with the superstructure removed. The differences between the solid line and the dashed line indicate the response caused by the inertial force of the superstructure. The small differences between the two lines reveal that the bending moment at the pile-head is largely dominated by ground motion. Figure 12 shows the distribution of the maximum bending moments of the piles along the depth. The differences also demonstrated that the response of the piles is to a large extent governed by kinematic interaction.

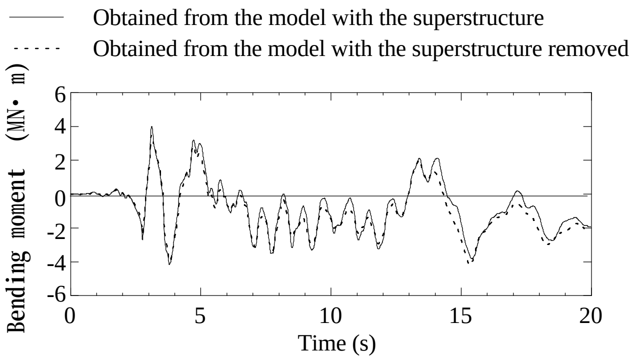



**Figure 11. Effect of inertial force of the superstructure on bending moment at the pile-head (PB3)**

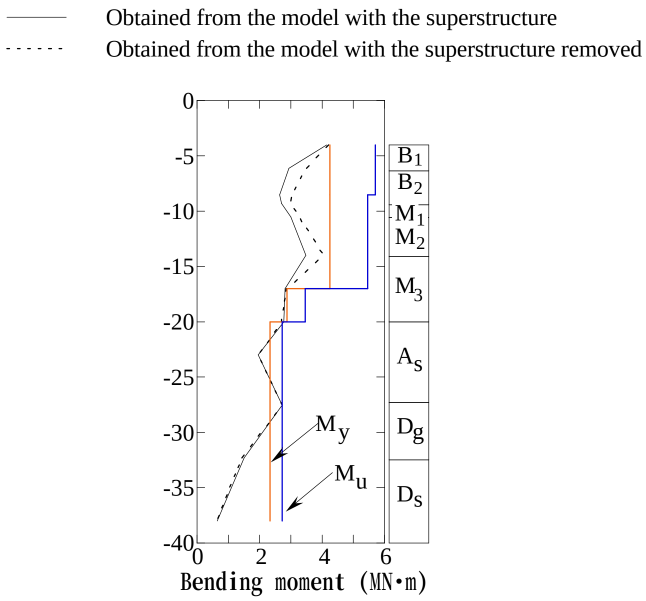



**Figure 12. Effect of inertial force of the superstructure on maximum bending moment of the pile (PB3)**

Meanwhile, in Figure 12, the maximum bending moment obtained from the analysis of the model with the superstructure removed exceeds that of the model with the superstructure at the depths from the pile-head to GL −17 m. It can be understood from Figure 11 that the bending moment of the model with the superstructure exceeds the bending moment of the model with the superstructure removed till about 13 seconds, however that of the model with the superstructure removed shifts to a negative value after 13 seconds and reaches the maximum value at 15.5 seconds. On account of the shift in the base line of the bending moment, the maximum bending moment of the model with the superstructure removed became larger than that of the model with the superstructure. This can be explained by the fact that differences in the nonlinear behavior of the ground occurred between both cases, and the nonlinear behavior of the ground in the model with the superstructure removed appeared stronger than in the model with the superstructure.

## LATERAL SPREADING ANALYSIS

The reduced shear modulus of the ground used in the lateral spreading analysis were defined based on the analysis [Tazoh et al., 1998] carried out to simulate the observed inground deformation caused by the lateral spreading of liquefied soil in Minami-Ashiyahama Island during the 1995 earthquake. The Poisson's ratio of the ground was defined using the method proposed by Kiku and Yoshida [Kiku and Yoshida, 1998] based on the theory of the invariant of the Poisson's ratio before and after the earthquake. As mentioned above, the dynamic response due to the shaking during the earthquake and the quasi-static response of the lateral spreading of the liquefied soil are evaluated separately in this study. In the lateral spreading analysis, the condition of damage in the piles caused by the shaking should be taken into account.

Figure 13 shows the deformation of the front side of the symmetrical plane of the model obtained by the lateral spreading analysis. The deformation has been drawn to 2.5 times actual scale. Soil layers M1 and M2 moved largely toward the river as a result of the movement of the liquefied soil layer M3 and a large settlement at the top of soil layer M1 was identified. The deformation modes of the ground and piles obtained from the analysis are in good agreement with those observed at the site. Figure 14 shows the distribution of the lateral displacements and bending moments along the depth of the PB3 pile. The dotted marks are the observed and estimated displacements of the pile obtained from the borehole camera survey. It can be recognized that there is some difference at the pile-head; however, the analytical results of the pile deformation in the deep part of the ground agree well with the observations.

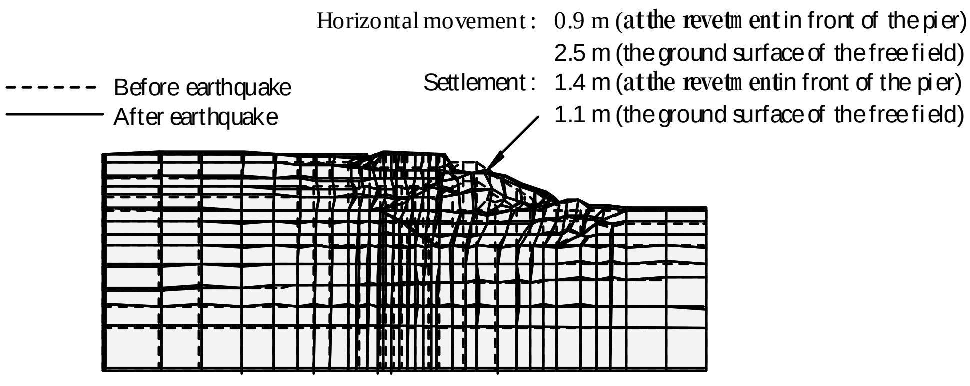
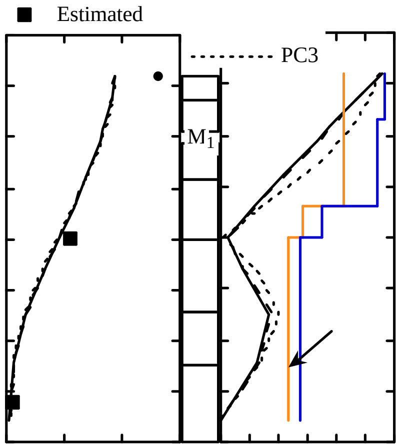



**Figure 13. The residual deformation obtained from the lateral spreading analysis (The deformation is drawn to 2.5 times actual scale)**




The maximum bending moments of piles PA3, PB3, and PC3 exceed the value of My at the above depths of GL −8 m and GL −10 m, respectively, and the maximum bending moment at the pile-head becomes larger than that obtained from the liquefaction analysis. This means that the damage at the pile-head, which occurred during the shaking, advanced as a result of the lateral spreading of the liquefied soil. The fact that many cracks at the above depths of GL −12 m were discovered during borehole camera investigations agrees well with the analytical result.

As shown in Figure 10, the maximum bending moment at the above depth of GL −12 m obtained from the soil liquefaction analysis was less than the value of My. However, the values at the same depths obtained from the lateral spreading analysis exceed the value of My. It can therefore be concluded that the cracks at the above depths of GL −12 m were caused by the lateral spreading of the liquefied soil. According to the results obtained from the soil liquefaction analysis, the maximum bending moments exceeded the value My at the depths of GL −17 m and GL −20 m, however the maximum bending moments at the same depths obtained from the lateral spreading analysis were less than the value of My. This phenomenon can be explained by the fact that the cracks which appeared at the depths of GL −17 m and GL −20 m were not caused by lateral spreading. It seems reasonable to conclude that the deformation of the piles due to the earthquake was aggravated by the lateral spreading of the liquefied soil.

**EXAMINATION OF EFFECTIVE REMEDIAL MEASURES**

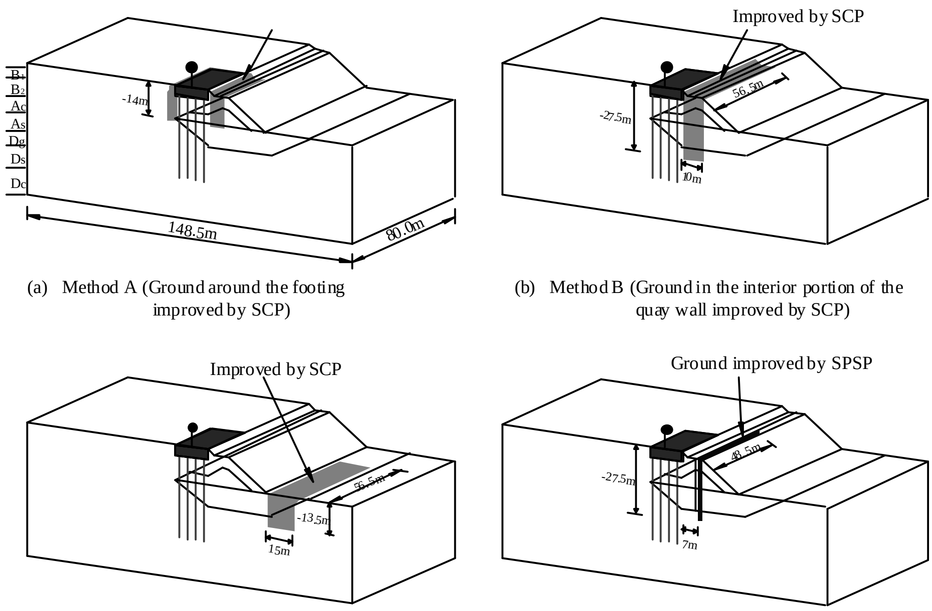
Remedial measures involving the adoption of sand-compaction piles (SCP) or steel-pipe-sheet piles (SPSP) were investigated in order to assess analytically the level to which damage can be mitigated. Four remedial measures, as shown in Figure 15, were examined in this study.

[Method A]: Soil improved by SCP in the periphery of the foundation of the pier, covering an area with a width of 5 m and a depth of 14 m, to the bottom of soil layer B2.

[Method B]: Soil improved by SCP between the foundation of the pier and the quay wall, covering a rectangular parallelepiped area with a width of 10 m, a length of 56.5 m, and a depth of 27.5 m, to the bottom of the liquefiable sandy soil layer As.

[Method C]: Soil improved by SCP in front of the quay wall, covering a rectangular parallelepiped area with a width of 15 m, a length of 56.5 m, and a depth of 13.5 m.

[Method D]: Installation of SPSP between the foundation of the pier and the quay wall, consisting of a pipe pile with a diameter of 1.2 m and a thickness of 16 mm, covering a line with a length of 48.5 m and a depth of 27.5 m.




(a) Method A (Ground around the footing improved by SCP)

(b) Method B (Ground in the interior portion of the quay wall improved by SCP)

(c) Method C (Ground on the waterfront side of the quay wall improved by SCP)

(d) Method D (Ground improved by SPSP)

**Figure 15. Four remedial measures to mitigate seismic damage**

Figure 16 indicates the maximum horizontal displacements and bending moments in the pile obtained from soil liquefaction analysis when each of the four remedial measures is utilized. For method A, the horizontal displacement and the bending moment in the pile do not differ from those of the unimproved model. For methods B, C, and D, the horizontal displacements are considerably smaller than that of the unimproved model. There is no difference in the bending moment in the pile between that in the model employing method C and that in the unimproved model. Methods B and D reduce the bending moment in the pile except for at the pile-head. The bending moment in the pile between soil layers As and Dg does not exceed the ultimate moment capacity of the pile and the collapse of the pile in the ground is avoided.

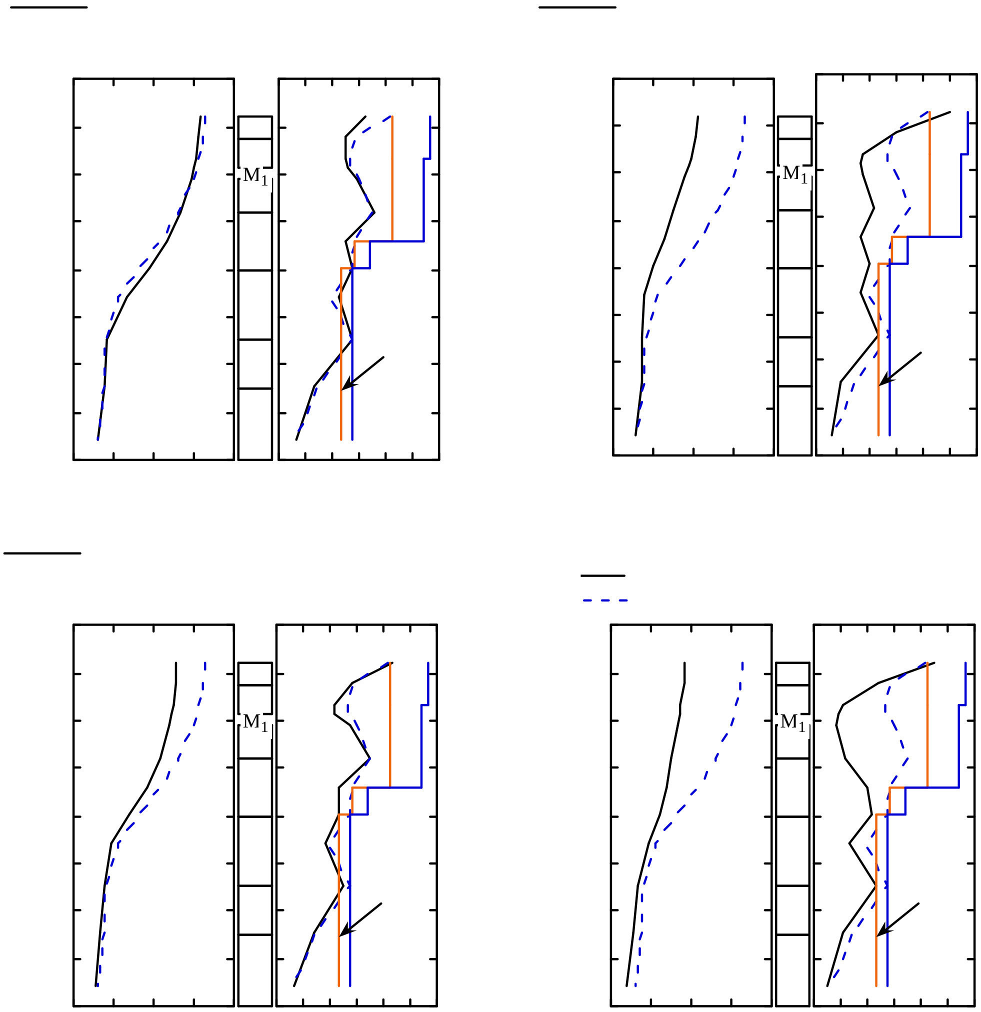
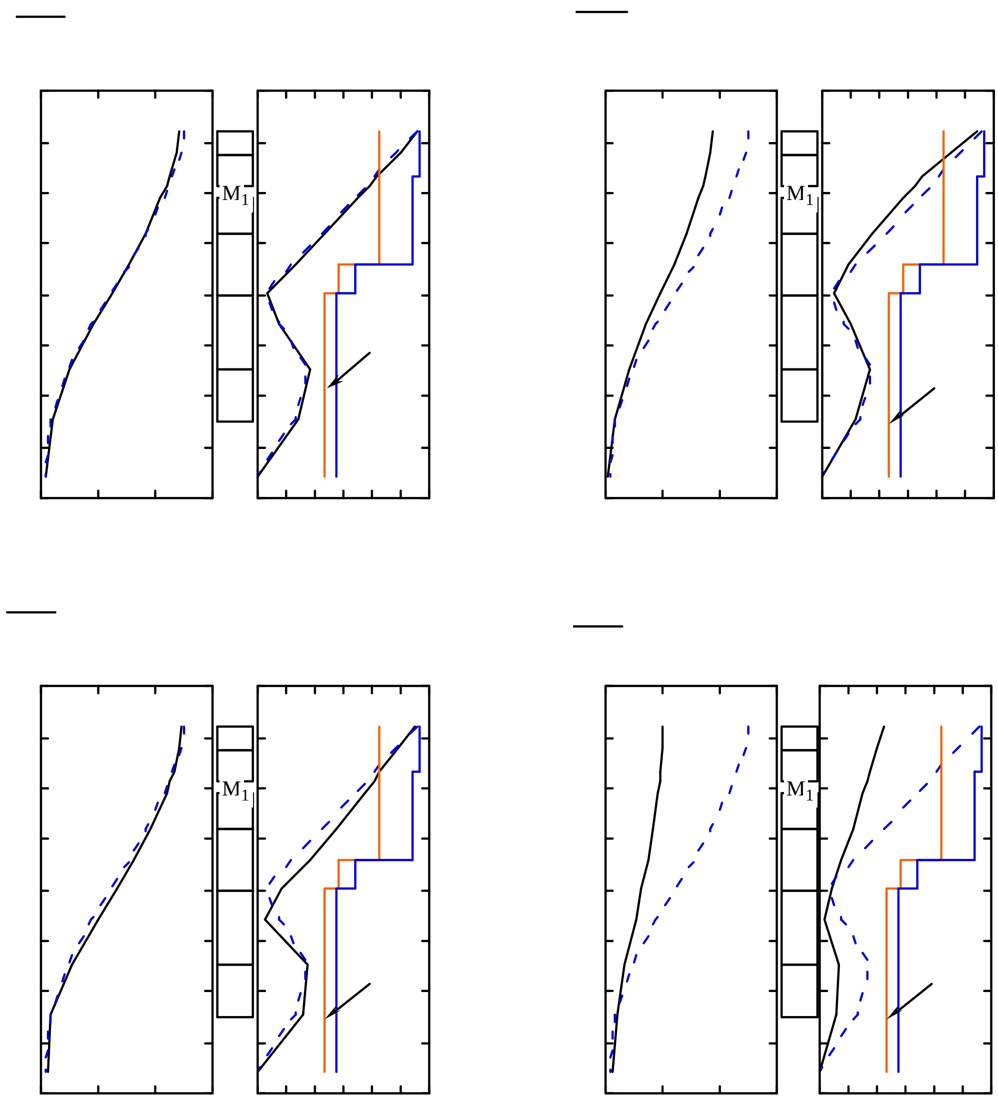



Figure 17 illustrates the horizontal displacements and bending moments in the pile obtained from lateral spreading analysis. The horizontal displacements and bending moments in the pile employing methods A and C are almost identical to those of the unimproved model. The bending moment in the pile-head employing method B exceeds the yield moment capacity of the pile. The horizontal displacement in the pile employing method D is 0.5 times that of the unimproved model. With this method (D), the bending moment along the pile is also reduced compared to the unimproved model, and does not exceed the yield moment capacity of the pile. It is tentatively concluded therefore that method D, with steel-pipe-sheet piles constructed between the foundation of the pier and the quay wall, is the most viable means of mitigating seismic damage.




**SEISMIC PERFORMANCE OF PILES WHEN THE NUMBER OF REINFORCING BARS IN THE PILES IS NOT DECREASED ALONG THEIR DEPTH**

In current design methods for road bridges in Japan, only the inertial force of superstructure is taken into account. Accordingly, the maximum bending moment will generally be produced at the pile-head, so that the quantity of reinforcing bars in piles is therefore decreased along the depth of piles for economic reasons. The reinforcing bars in the damaged piles mentioned above have been decreased as shown in Table 1. There is a strong possibility that if ground motion is taken into account in the design codes the number of reinforcing bars cannot be decreased along the pile. We considered it to be an interesting proposition to investigate the extent to which the seismic performance of the piles would increase if the number of reinforcing bars was maintained along their depth and therefore attempted to examine this hypothesis via practical investigation.

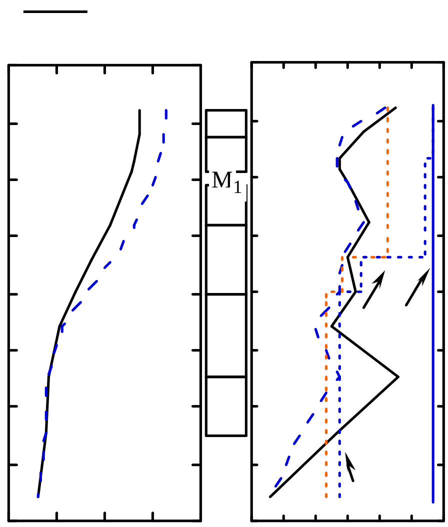
**Table 1. The reinforcing bars of the piles**

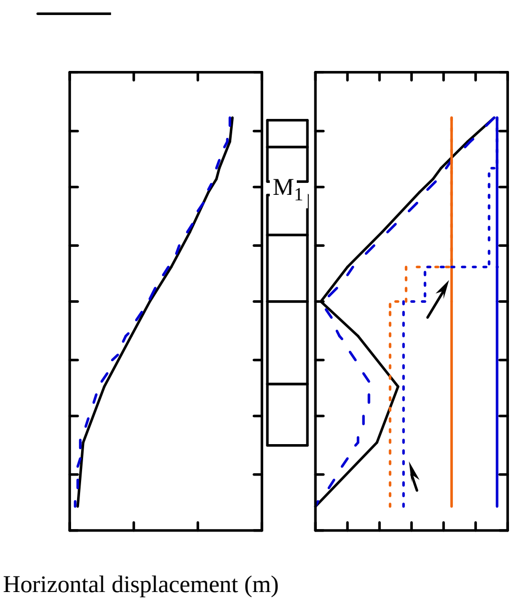
| Depth | Damaged piles with a decreased number of reinforcing bars | | | | Piles with a non-decreased number of reinforcing bars | | | |
|---|---|---|---|---|---|---|---|---|
| | Main reinforcement | | Hoop | | Main reinforcement | | Hoop | |
| | Nominal diameter | Number | Nominal diameter | Space between hoops | Nominal diameter | Number | Nominal diameter | Space between hoops |
| GL -4.0m ~ -8.6m | D32 | 30 | D19 | 15cm | D32 | 30 | D19 | 15cm |
| -8.6m ~ -17.0m | D32 | 30 | D19 | 30cm | | | | |
| -17.0m ~ -18.3m | D32 | 30 | D19 | 50cm | | | | |
| -18.3m ~ -20.9m | D32 | 15 | D19 | 50cm | | | | |
| -20.9m ~ | D25 | 15 | D19 | 50cm | | | | |

**Table 2. Maximum accelerations and displacements for the two cases**

| Bar arrangements | Pier | | Footing | |
|---|---|---|---|---|
| | Acceleration ($m/s^2$) | Displacement (m) | Acceleration ($m/s^2$) | Displacement (m) |
| Non-decreased reinforcing bar | 3.28 | 0.28 | 2.50 | 0.28 |
| Decreased reinforcing bar | 2.71 | 0.33 | 2.18 | 0.33 |

Table 2 shows the comparison of the maximum responses of the pier for the two cases: 1) where the number of reinforcing bars is decreased and 2) where the number of reinforcing bars is maintained along the depth of the piles. The maximum displacements of the superstructure and the base of the pier for the latter case became smaller, while the maximum accelerations of the former case increased in size. For this reason it can be understood that the rigidity level of the piles for the latter case would not decrease because the number of yielding points on the reinforcing bars is decreased. Figure 18 shows the distribution of the maximum lateral displacement and bending moments of the piles. Myn and Mun represent the yield moment of the reinforcing bars and the ultimate bending moment for the case in which the number of reinforcing bars is maintained along the depth of the piles. In this case, the maximum displacement became smaller at the above depths of GL –20 m, however there were no major differences at the depths below GL –20 m. As for the maximum bending moments, those at the depths from GL –17 m to GL –20 m exceeded the value of My, and that at the depth of GL –27.5 m reached the value of Mu for the case of the damaged piles. On the other hand, the maximum bending moments at the same depths for the case in which the number of reinforcing bars was maintained became less than the value of Mun. Figure 19 shows the results of the lateral spreading analysis. It can be seen that the displacement of the pile can not be decreased by adopting piles with a constant number of reinforcing bars along their depth, and also that the bending moments at the pile-head in both cases reached the same value; effectively the value of Mu or Mun (Mu and Mun are the same in the upper section of both cases). On the other hand, in the case where the number of reinforcing bars is maintained along the depth of the piles, there is no point that exceeds the value of Mun along the depths of the pile except at the pile-head. It can therefore be concluded that the level of damage to the piles will decrease by adopting this type of piles, however the maximum bending moment of the pile at the pile-head and the boundary of the liquefied and the un-liquefied soil layers will exceed the value of Myn. The bending moment at the pile-head obtained from the lateral spreading analysis reached the value of Mu and Mun in both cases, so that in both cases it will be necessary to reinforce the pile-heads, by attaching a steel-pipe-pile to the outside of the cast-in-place concrete pile for example.




**. Effect of number of the reinforcing bars on displacements**
**nents of the pile (PB3) obtained from the lateral spreading a**

**SUMMARY OF THE DAMAGE ANALYSIS**

The main findings obtained from the damage analysis can be summarized as follows:

(1) The damage to the pile foundation was mainly caused by kinematic interaction during the main shock of the earthquake. It was expanded by the lateral spreading of the liquefied soil, which occurred in succession after the main shock, causing large residual deformation to the piles.

(2) Remedial measures involving the adoption of sand-compaction piles or steel-pipe-sheet piles were examined. It was found that the steel-pipe-sheet piles constructed between the foundation of the pier and the quay wall can withstand strong motion, soil liquefaction, and the lateral spreading induced therefrom.

(3) The damage level can be decreased by adopting piles with a constant number of reinforcing bars along their depth, however it is still necessary to strengthen the pile-head.

## SEISMIC ISSUES OF PILE FOUNDATIONS REVEALED BY THE 1995 EARTHQUAKE

**SEISMIC DAMAGE**

Investigating the causes of seismic damage is the root of seismic engineering. For pile foundations, many causative factors have been elucidated; inertial force of the superstructure, ground motion, soil liquefaction, lateral spreading induced by soil liquefaction, qualitative or superannuated problems of the piles, imperfections in design codes, mistakes in the design process or during construction, etc. It was concluded as a result of the damage analysis described above that the damage to the pile foundation was mainly caused by ground motion during the main shock of the earthquake, and that this was expanded by the lateral spreading of the liquefied soil, which occurred in succession after the main shock, causing large residual deformation to the piles. The point to be emphasized is that the damage to the pile foundation was caused by ground motion, namely kinematic interaction, which is not taken into account in the seismic design codes used in Japan, and further that the damage was not the result of a single causative factor of a moment, but due to the multiple causes of ground motion, soil liquefaction, and lateral spreading induced by the soil liquefaction over time.

Since the 1995 earthquake, the phenomena of lateral spreading induced by soil liquefaction has been actively discussed because numerous reports have indicated that the lateral spreading of liquefied soil intensified the damage to various structures. The severe damage to pile foundations is considered to have occurred mainly as a result of lateral spreading induced by soil liquefaction, and therefore the force created exclusively by the lateral spreading of liquefied soil has been focused on and evaluated. However, in the case where the damage is the result of multiple causes, the force must be overestimated. It goes without saying that in order to establish rational design codes it is crucial to correctly interpret the cause of the damage.

The indication that kinematic interaction remarkably affects the damage to pile foundations is not new. Some reports [Horikoshi and Ohtsu, 1996, Structural Engineering Committee of Architectural Institute of Japan, 2000] clarifying the fact that the damage is the result of kinematic interaction have been published since the 1995 earthquake. The damage to pile foundations occurred as the result of under-construction of the superstructure, nonetheless, there were many pile foundations that sustained no damage in the ruinous earthquake during the 1995 earthquake. What elements in the design codes applied to these latter pile foundations prevented damage despite the greater force than that used in the design acting on them? Is it acceptable to assume that some safety factor was hidden in the design or construction process? Is it natural that damage will occur when a force larger than that defined in the design is applied to the structures? Should the design be deemed to be over-design if damage does not occur even when a larger force than that used in the design is applied to structures? We have been actively seeking adequate answers to these questions.

It should be noted that not all types of pile foundations sustained major damage. The incidence of damage to the pile foundations of civil engineering structures was relatively low. Should pile foundations be designed so that no damage would be sustained or so that a certain level of damage would be acceptable if a very severe earthquake occurred? How much and where in the piles is damage acceptable if some damage could be permitted? It should be pointed out that the damage caused by the 1995 earthquake needs to be re-evaluated based on the perspective of acceptable and un-acceptable levels of damage.

### SEISMIC DESIGN METHOD OF PILE FOUNDATIONS

The numerical examination mentioned above revealed that severe damage would be mitigated if the number of reinforcing bars were maintained along the whole length of the piles. Our point is that it is a matter of extreme urgency to take the effect of the kinematic interaction of pile foundations into account when creating seismic design codes. For this reason, it is necessary to conduct more specific investigation into how damage will occur and the affect it will have on the seismic safety of the superstructure if the kinematic interaction is not taken into account. There are many issues that require resolution if a reasonable design method is to be established, for example, to the extent and nature of the inertial force and ground deformation applying to the piles. It is also important to investigate the extent to which the performance and cost of the piles will be increased when kinematic interaction is taken into account. New technologies to decrease the construction costs of pile foundations will also become necessary.

## REMARKS

Since the 1995 Great Hanshin Earthquake, major progress has been made in earthquake engineering research based on the damage exposed in Kobe. It should be noted that the damage caused by the 1995 earthquake might not encompass all foreseeable types of damage however. To be more precise, other, different phenomena may be generated in the event of a major earthquake occurring in another metropolitan area such as Tokyo. A comprehensive overview should therefore be adopted when investigating known and potential causes of seismic damage.

## ACKNOWLEDGEMENTS

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