ANALYSIS OF THE DAMAGE TO THE PILE FOUNDATION OF A HIGHWAY BRIDGE CAUSED BY SOIL LIQUEFACTION AND ITS LATERAL SPREAD DUE TO THE 1995 GREAT HANSHIN EARTHQUAKE

Takashi TAZOH[1], Akira OHTSUKI[2], Masaki FUCHIMOTO[3], Atsushi NANJO[4], Furitsu YASUDA[5], Yasuo FUJII[6], Akinori NAKAHIRA[7] And Choji KURODA[8]

SUMMARY

The damage to the pile foundation of a bridge on the Hanshin Expressway caused by the 1995 Great Hanshin Earthquake was studied, and the cause of the damage was examined by means of the three dimensional finite element method taking into account soil liquefaction and the lateral spreading of the liquefied soil. A 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. It was determined by the analyses that the damage was mainly caused by the inertial force of the superstructure and the ground motion during the earthquake, and the damage 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. 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, liquefaction, and the lateral spreading it induces.

INTRODUCTION

Serious damage to pile foundations was caused by the Great Hanshin Earthquake which occurred on January 17, 1995. The extent of the damage was made infinitely worse by soil liquefaction and the lateral spreading it induced. A bridge on the Hanshin Expressway Route No. 5, which had been constructed on young reclaimed land of soft soil deposit, suffered heavy damage, on the verge of causing its girders to fall during the earthquake. There arose an urgent need to examine the cause of the damage and to study remedial measures to mitigate similar damage in Japan because many key structures have been constructed on reclaimed land as was this bridge.

OUTLINE OF THE BRIDGE AND DAMAGE TO THE PIER

The damaged bridge has three spans of Gerber-steel-box girders 310 meters in length which cross over 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 steel-rigid-framed pier on Minami-Ashiyahama Island is supported by 56 piles (4 rows x 14 columns) which are cast-in-place concrete piles having 34 meters in length and 1.5 meters in diameter. The soil deposit consists of the reclaimed layer (GL 0 ~ –14 m) composed mainly of gravel, the alluvial clayey soil layer (GL –14 ~ –20 m), the alluvial sandy soil layer (GL –20 ~ –28 m), and the diluvium sandy soil layer (under GL –28 m). The ends of the piles are embedded in the diluvium sandy soil layer. Figure 1 illustrates the residual deformation of the pier caused by the earthquake. The pier moved about one meter laterally toward the river. The revetment moved about three meters laterally in the same direction and

[1] *Shimizu Corporation, Institute of Technology, Tokyo, Japan (Email:tazoh@sit.shimz.co.jp, Fax:+81-3-3820-5959)*
[2] *Shimizu Corporation, Institute of Technology, Tokyo, Japan*
[3] *Shimizu Corporation, Institute of Technology, Tokyo, Japan*
[4] *Hanshin Expressway Public Corporation, Osaka, Japan*
[5] *Hanshin Expressway Public Corporation, Osaka, Japan*
[6] *Hanshin Expressway Public Corporation, Osaka, Japan*
[7] *C. T. I. Engineering Co., Ltd., Japan*
[8] *C. T. I. Engineering Co., Ltd., Japan*

subsided by more than one meter. The occurrence of soil liquefaction around the bridge was identified by evidence of boiled sand flow on the surface of the surrounding ground. Photograph 1 shows the ground failure around the pier. The pier located on Nishinomiyahama Island moved 0.6 meters laterally toward the river.

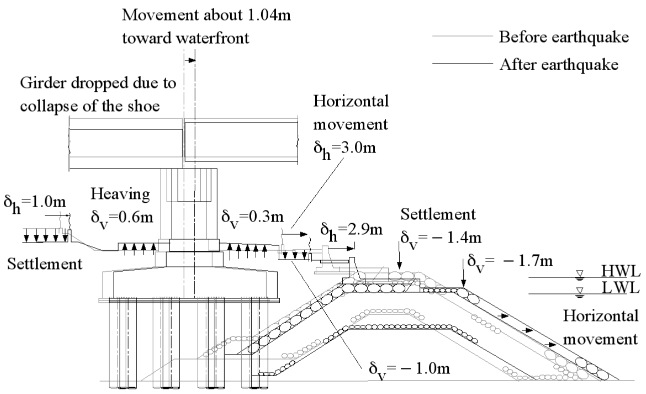

Figure 1 Residual horizontal displacements and settlements of the pier and the revetment (The cross section is viewed from the south)

Photograph 1 The damage to the ground in front of the pier (Taken from the north side of the pier)

3. 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, the integrity sonic test utilizing elastic waves, and the indirect inspection utilizing a borehole camera. Figure 2 shows the result of the investigation using the borehole camera. 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 was different in boreholes A and B. Many cracks appeared between the pile-head and 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.

ANALYTICAL PROCEDURES

In this study, separate evaluations are made 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. The procedure proposed by Ohtsuki et al. [Ohtsuki et al., 1998] is used in the soil liquefaction analysis. The modified Ramberg-Osgood and the bowl models [Fukutake and Matsuoka, 1989] are adopted for the stress-strain and strain-dilatancy relationships in the procedure. Initial stresses in the ground are out of balance because the shear stiffness of the liquefied soil deposits, which have a high potential for excess pore water pressure, decreases 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 by static analysis due to gravity force, under the condition that the stiffness values of the liquefied soil deposits decrease according to the accumulation of excess pore water pressure, which data was obtained from the soil liquefaction analysis.

ANALYTICAL MODEL

Figure 3 shows the numerical model (a half model is used) of the pier, pile foundation, and ground for three dimensional finite element analysis. The number of nodes and elements are 10,539 and 9,557, respectively. The pier was modeled by a linear beam. The modified Takeda model [Fuchimoto and Ohtsuki, 1997] was used for analyzing the nonlinear behavior of the piles. The input motion shown in Figure 4, which is observed at GL –83 m in Port Island, where the distance from the pier is about 12 km, was used for the analysis. 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.

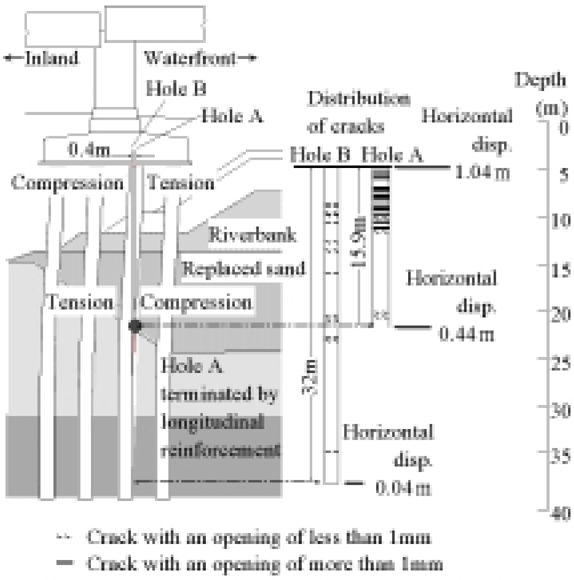

Figure 2 Cracks and horizontal displacements in the pile observed by the borehole camera (The cross section is viewed from the south)

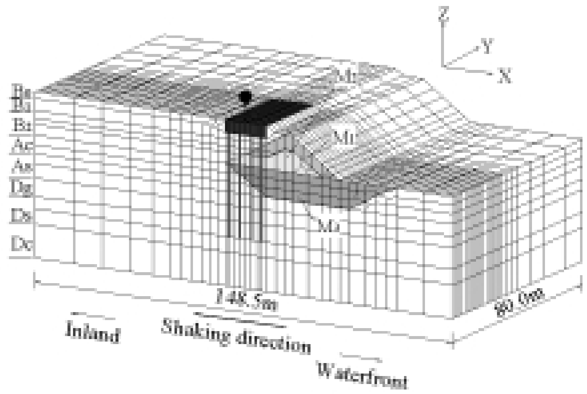

Figure 3 Three dimensional analytical model for the soil-piles-pier system (The front side is the symmetrical plane)

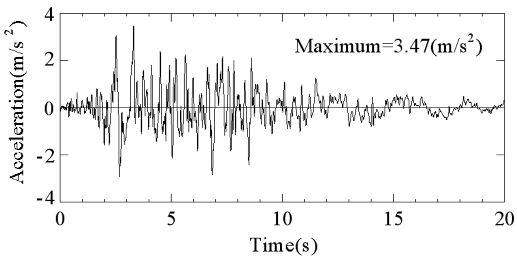

Figure 4 Input motion (The records of two components observed at Port Island at a depth of 83 m are transferred to the longitudinal and the orthogonal components of the pier for the analytical model)

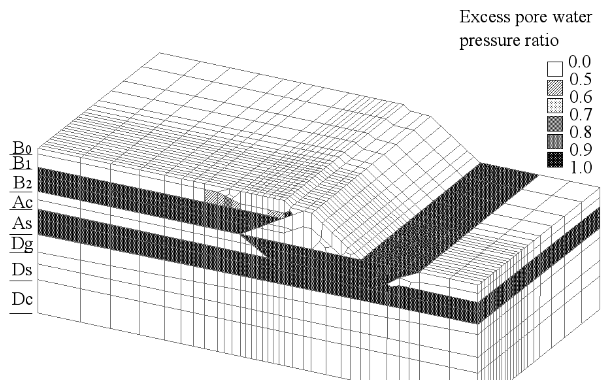

Figure 5 Excess pore water pressure ratio obtained from the soil liquefaction analysis

SOIL LIQUEFACTION ANALYSIS

Figure 5 represents the maximum value of the excess pore water pressure ratio obtained by the soil liquefaction analysis. It could be detected that the whole region of the layers B_2, As, M_3, and a part of the layer B_1 were liquefied since the excess pore water pressure ratio exceeded 0.95. Figure 6 shows the time histories of the excess pore water pressure ratio of B_2 (GL –8.5 ~ –10.5 m) and As (GL –20 ~ –24m) which are the underlying 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 happened three or four seconds after the occurrence of the earthquake according to the peak of the input motion. The predominant period of the acceleration after the peak became longer, which was caused by the liquefaction of the layer As after four seconds. The predominant period of the displacement also became longer after four seconds. The time histories of the bending moments at the pile-head of three piles (A3, B3, and C3) are shown in Figure 7. The pile B3 corresponds to the pile investigated by the borehole camera. My and Mu in the figure represent the yield moment of reinforcement and the ultimate bending moment of the pile, respectively. The peaks of the bending moment close to the My appeared at 3 ~ 4 seconds, 7 ~ 9 seconds, and 15 seconds. As for the relationship of the peaks with the bending moment and the acceleration/displacement, and the peak of the bending moment at 3 ~ 4 seconds corresponds to the peaks of the acceleration and displacement, the peaks at 7 ~ 9 and 15 seconds correspond to the peaks of the displacement. It can be concluded that the peak of the bending moment at 3 ~ 4 seconds occurred due to the inertial force of the superstructure and the ground motion during the main shock of the earthquake, and the occurrence of peaks at 7 ~ 9 seconds and 15 seconds was influenced by the ground motion affected by soil liquefaction.

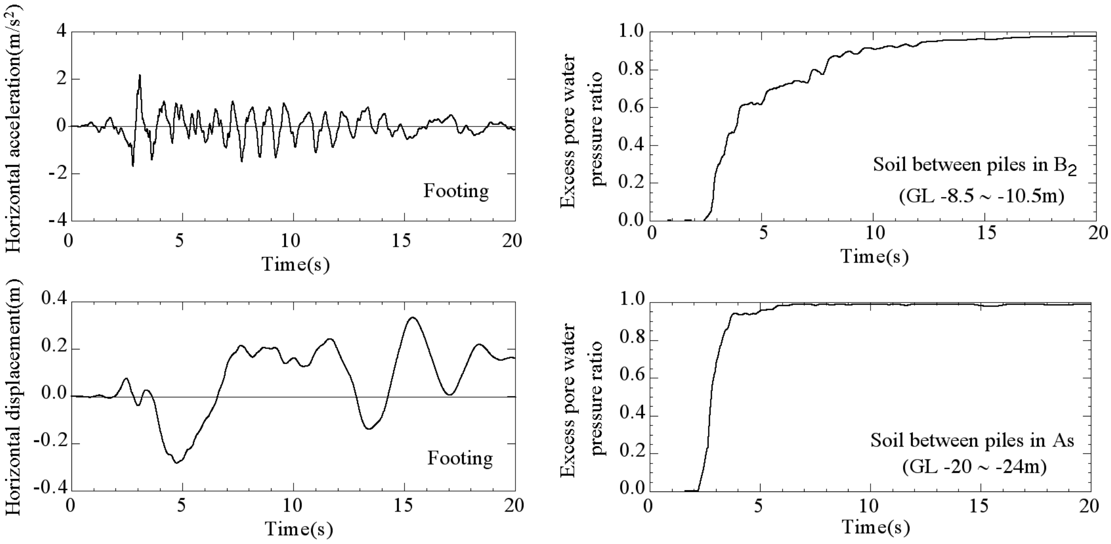

Figure 6 Time histories of the acceleration and the displacement at the footing of the pier and the excess pore watre pressure ratio in B2 and As layres

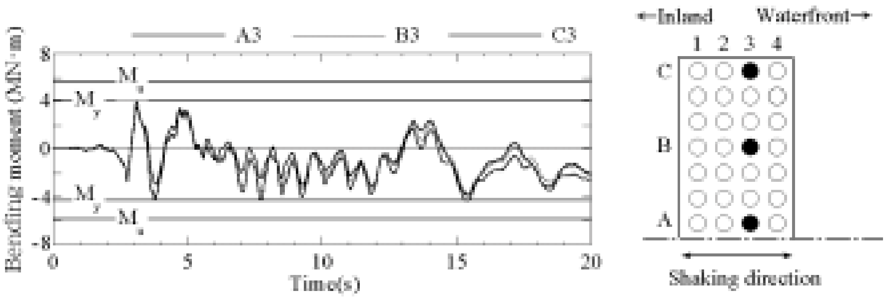

Figure 7 Time history of the bending moments at the pile-head of the piles A3, B3, and C3

Figure 8 illustrates the distribution of the lateral displacements and bending moments of the three piles. The piles were bent largely in the liquefied layer As. The bending moments became significantly large at the upper and lower boundaries (GL –20 m and –27.5 m) of the layer As, since the soil stiffness changes greatly at these boundaries. The maximum bending moments which exceeded the yield value of the pile obtained by the soil liquefaction analysis corresponded to the positions of the observed cracks on the piles at the pile-head, GL –17

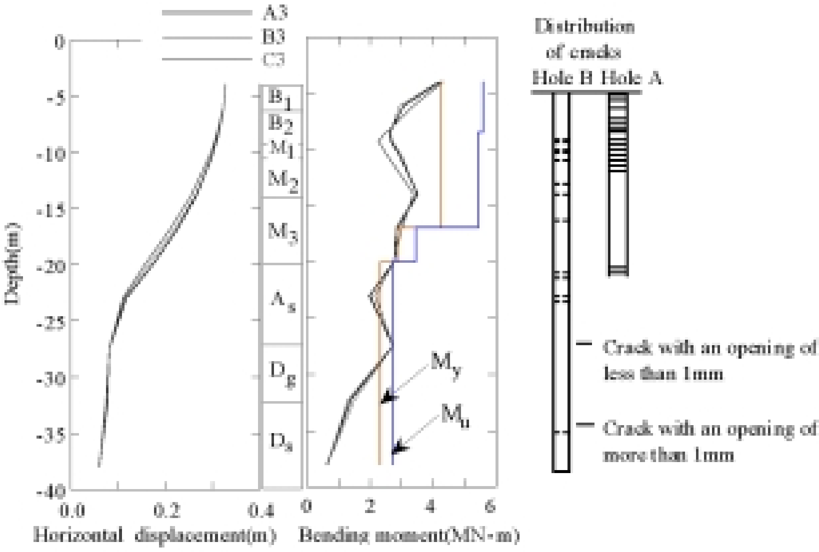

Figure 8 Maximum horizontal displacements and bending moments of the piles (A3, B3, and C3) obtained from the soil liquefaction analysis

m, and GL –20 m. Consequently, it can be concluded that the cause of these cracks was the inertial force of the superstructure and the ground motion during the shaking.

LATERAL SPREADING ANALYSIS

The reduced shear modulus and Poisson's ratio of the ground used in the lateral spreading analysis were defined based on the simulation analysis [Tazoh et al., 1998] of the observed inground deformation caused by the lateral spreading of liquefied soil in Minami-Ashiyahama Island during the 1995 earthquake. As mentioned above, the dynamic response due to the shaking during the earthquake and the quasi-static response of lateral spreading of the liquefied soil are evaluated separately in this study. In the lateral spreading analysis, the damaged condition in the piles caused by the shaking should be taken into account. Figure 9 shows the distribution of the lateral displacements and bending moments along the depth of the B3 pile. The dotted marks are the observed points of the deformed 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 inground deformation agree with the observations. It is concluded that the deformation of the piles damaged due to the shaking of the earthquake was aggravated by the lateral spreading of the liquefied soil.

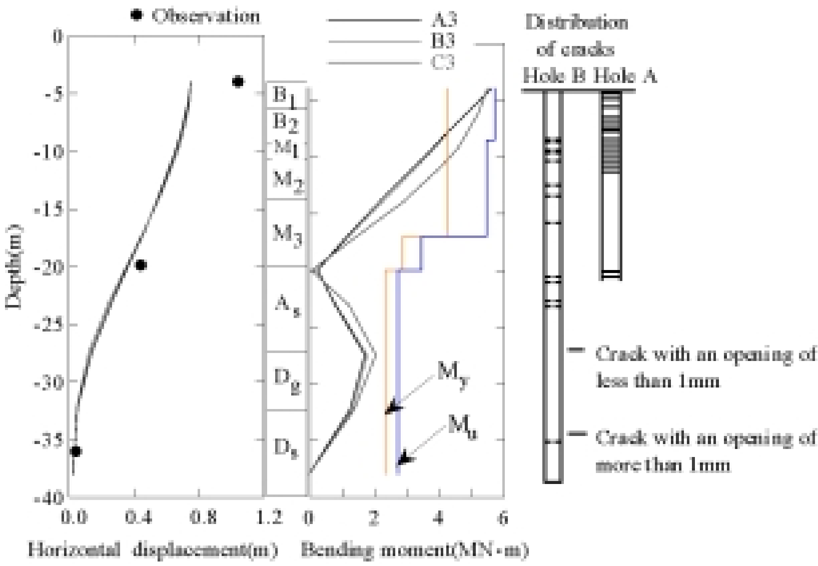

Figure 9 Horizontal displacements and bending moments of the piles (A3, B3, and C3) obtained from the lateral spreading analysis

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 the damage could be mitigated.

[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 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 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.

Figure 10 indicates the maximum horizontal displacements and bending moments in the pile obtained by the soil liquefaction analysis when the four remedial measures are 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

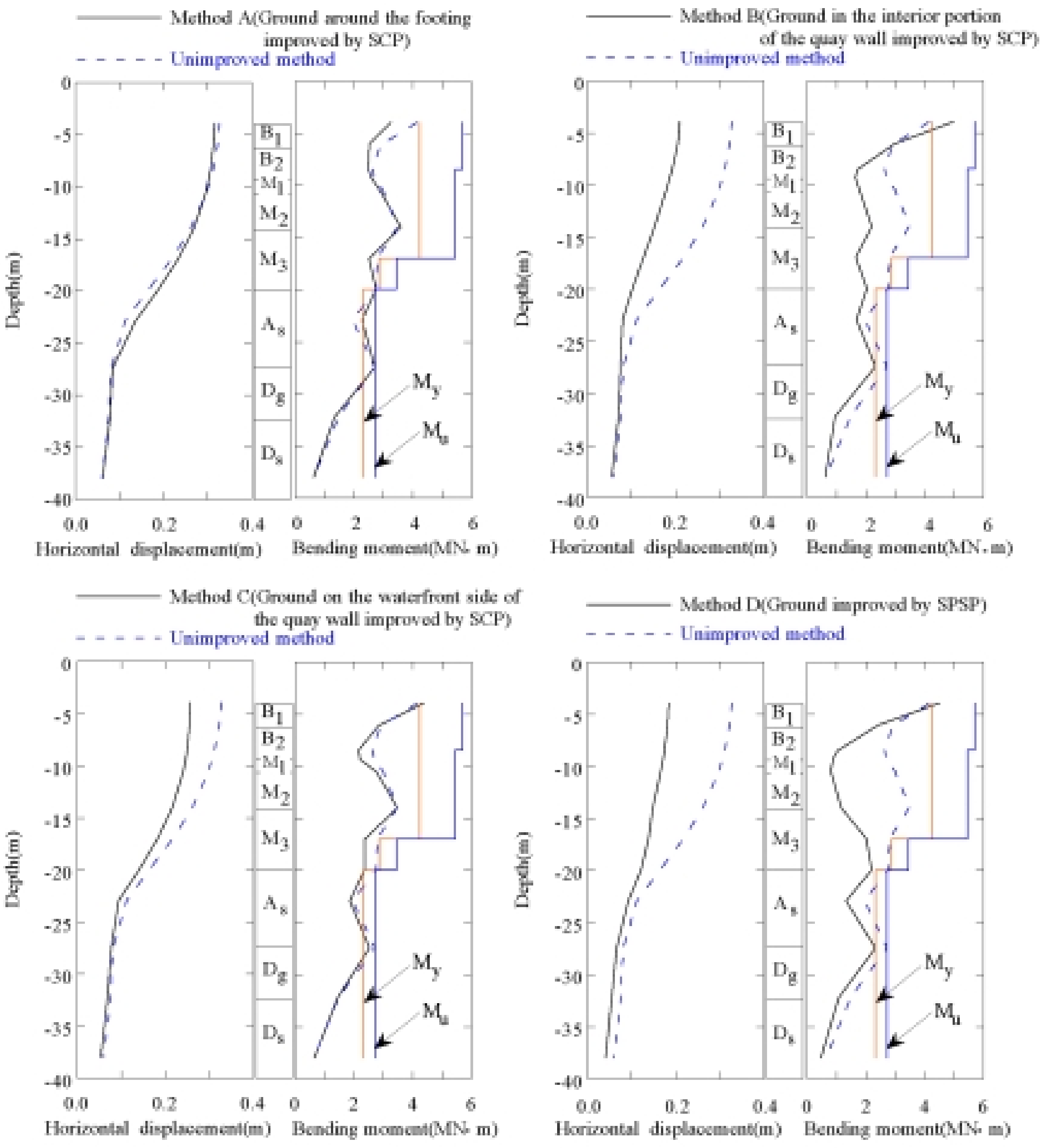

Figure 10 Maximum horizontal displacements and bending moments of the piles (B3) obtained from the soil liquefaction analysis for the four methods (A~D)

the horizontal displacements are considerably smaller than that of the unimproved model. There is no difference in the bending moment in the model employing method C and the unimproved model. Methods B and D reduce the bending moment in the pile except for the pile-head. The bending moment in the pile between the 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 11 illustrates the horizontal displacements and bending moments in the pile obtained by the lateral spreading analysis. The horizontal displacements and bending moments in the pile with methods A and C are almost the same as those of the unimproved model. The bending moment in the pile-head with method B exceeds the yield moment capacity of the pile. The horizontal displacement in the pile with method D is 0.5 times that of the unimproved model. The bending moment along the pile with method D is reduced compared to the unimproved model, and it does not exceed the yield moment capacity of the pile.

It is tentatively concluded that method D, with steel-pipe-sheet piles constructed between the foundation of the pier and the quay wall, can be a most viable means of mitigation.

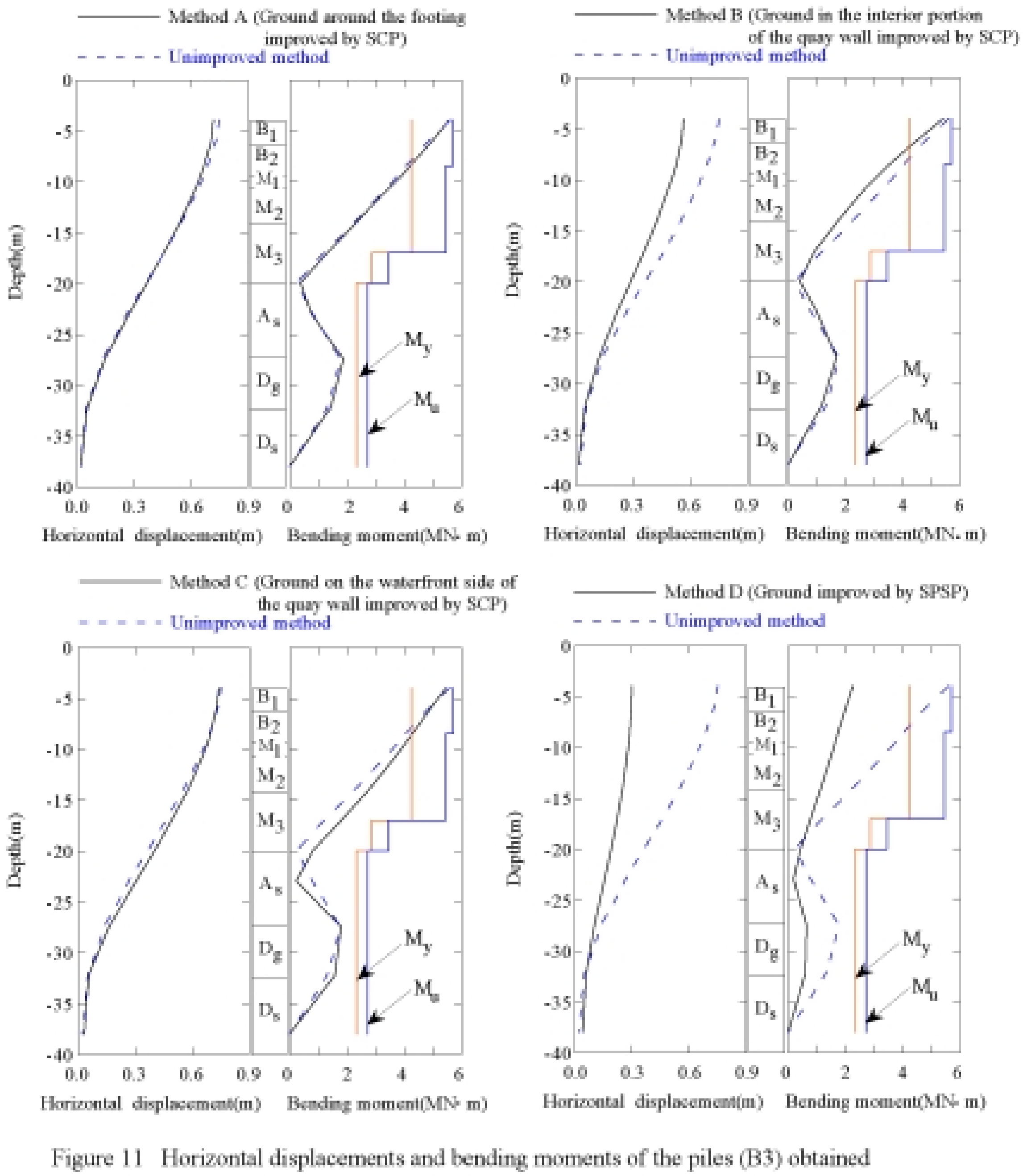

Figure 11 Horizontal displacements and bending moments of the piles (B3) obtained from the lateral spreading analysis for the four methods (A~D)

CONCLUSION

The main findings can be summarized as follows:

(1) The maximum bending moments at some points, the pile-head, and pile–inground exceeded the yield moment of the pile, and the positions of the observed cracks on the piles corresponded to these points.

(2) The damage to the pile foundation was mainly caused by soil liquefaction 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.

(3) 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, liquefaction, and the lateral spreading thereby induced.

ACKNOWLEDGEMENT

The authors are indebted to Associate Professor Kenji Kosa of the Kyusyu Institute of Technology for his expert advice on the technical aspects of this research.

REFERENCES

Ohtsuki, A., Fukutake, K. and Sato, M. (1998) : Analytical and Centrifuge Studies of Pile Groups in Liquefiable Soil Before and After Site Remediation, Earthquake Engineering and Structural Dynamics, Vol. 27, pp.1-14.

Fukutake, K. and Matsuoka, H. (1989) : A Unified Law for Dilatancy under Multi-directional Simple Shearing, Proceedings of Japan Society of Civil Engineers, No.412 / III-12, pp.143-151 (in Japanese).

Fuchimoto, M. and Ohtsuki, A. (1997) : Non-Linear Response of Group Pile Foundation Assessed by Three Dimensional Effective Stress Analysis, Proceeding of the Thirty-second Japan National Conference of Geotechnical Engineering, Vol.1, pp.969-970 (in Japanese).

Tazoh, T., Ohtsuki, A., Fuchimoto, M., Kishida, S., Nanjo, A., Yasuda, F., Kosa, K., Fujii, Y., Tamba, Y., Nakahira, A. and Kuroda, C. (1998) : Measurement and Analysis of In-ground Horizontal Displacements at a Site of Soil Liquefaction and Lateral Spread in Reclaimed Land Caused by the 1995 Great Hanshin Earthquake, Proceedings of the Eleventh European Conference on Earthquake Engineering.