PERMANENT DEFORMATION OF STEEL PIPE PILES PENETRATING COMPACTED FILL AT WHARF ON PORT ISLAND

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ABSTRACT

A pile inclinometer survey is conducted for the steel pipe piles supporting a four-story steel structure that survived the Hyogoken-Nambu earthquake, together with some other measurements, in order to clarify the degree of integrity of the building foundation. The building is located on a wharf surrounded by the sea on three sides. At the site, fill deposits about 20 m thick have been compacted by the vibro-rod compaction method. Subsurface investigations are also conducted for the compacted fill and underlying soil deposits.

The investigations reveal the following: (1) ground subsidence due to the earthquake is estimated to be 30 to 100 mm; (2) the compacted fill has 60 to 80 percent greater resistance to liquefaction than untreated fill; (3) piles about 40 m long have been deformed not only above the interface between the reclaimed fill and the underlying alluvial clay layer (Ma13 layer), but also at the lower interface of the Ma13 layer; (4) the pile heads are judged to have been moved by at most 344 mm in the north-northeast to northeast direction.

The moved distance and direction of the pile heads are explained on the basis of the compiled data regarding the relationships among the moved distance of pile-supported buildings, the distance between the buildings and quay walls, the lateral displacement of the quay wall crests and the thickness of the reclaimed fill.

Key words: compaction, lateral spreading, liquefaction, permanent deformation, pile, pile inclinometer survey, reclaimed fill, sampling, settlement (IGC: E8)

INTRODUCTION

In the 1995 Hyogoken-Nambu earthquake, two kinds of liquefaction remediation (remedial measures against liquefaction) were proved to be quite effective in helping pile-supported buildings survive the destructive earthquake. One way involves having piles installed in ground improved by vibratory compaction methods. The other involves surrounding the piles with solidified soil walls or diaphragm walls.

On Port Island, pile-supported buildings performed well at sites treated by the sand drain method, although this method was not primarily liquefaction remediation but remedial measures for facilitating consolidation settlement. This is probably due to the fact that the strong vertical vibration of the casing for penetrating the 2 or 3 m thick stiff slag layer situated around the lowest part of the fill densified the surrounding fill deposits during the

casing-installation operation. This unexpected effect can be considered to have served the purpose of restricting ground deformation within allowable limits, to lead to a reduction in damage to the buildings located in the central city zone of the island.

On Port Island, there was also a pile-supported building for which liquefaction remediation was positively taken before the earthquake, unlike the above-mentioned case. At the site, the fill deposits had been compacted using the vibro-rod compaction method originated by Saito (Saito, 1977). The building was located at the southeast corner of Port Island (Phase I), and situated on a wharf surrounded by the sea on three sides as shown in Photo. 1. Television observation and a pile inclinometer survey were conducted for the piles supporting the building that survived the earthquake. Subsurface investigations were also conducted including the sampling by the in-situ freezing method.

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Photo. 1. Wharf where the building is located (middle part of photo.) (taken shortly after the quake by Geographical Survey Institute)

The objective of this paper is to clarify the degree of integrity (structural soundness) of the building foundation that survived the earthquake, including the deformation mode of the piles that had been driven through the compacted reclaimed fill and an underlying alluvial clay layer (locally called Ma13 layer) to a bearing stratum, and to discuss the permanent deformation of the soil deposits under the special condition of the location, "wharf".

OUTLINE OF BUILDINGS AND SOIL CONDITIONS BEFORE AND AFTER EARTHQUAKE

Soil Conditions Before Ground Improvement

The reclamation of the site was started in 1970 and was completed in 1981 after about 10 years of filling-up with 8 year interruption. Around the end of the reclamation works, a subsurface investigation was performed at 4 points below or near the footprints of the buildings to be built, as shown by Nos. 1 to 4 in Fig. 1. Figure 2(a) shows the boring logs obtained. SPT *N*-values of the reclaimed fill deposits 18 m thick range between 5 and 12. Note that the fill deposits also show the quite higher *N*-values caused by the striking of a spoon sampler against boulders etc. The upper part of the alternating layers overlaid by the Ma13 layer consists mainly of sandy soil layers containing a fines content ranging from 10 to 30 percent with *N*-values between 10 and 20.

Liquefaction Remediation Adopted

As the fill had low *N*-values and the buildings were to

Fig. 1. Locations of borings, sampling and buildings (treated site by vibro-rod compaction)

Fig. 2. *N*-value profiles: (a) pre-treatment, (b) post-earthquake

be built as soon as the reclamation was completed, it was decided that the fill deposits would be improved by the vibro-rod compaction method. A vibratory steel pile hammer (vibro-rod) with H-shaped cross section, shown in Fig. 3, was used in the works (Tanahashi et al., 1987). It was devised to increase the effectiveness of compaction by attaching projected steel plate wings. After the vibro-rod was driven to a depth of 19 m with the help of compressed air jetting, the withdrawing-driving operation process was repeated until the tip of the vibro-rod reached the ground surface, as shown in Fig. 4. The fill was treated over its almost full 19 m thickness using a square pattern with 2.6 m spacings. As it is customary to extend the treatment zone laterally beyond the building footprints by a distance of half the depth of treatment, the vibro-rod treatment was extended a distance of 10 m around the buildings to be built, as shown in Fig. 1. The total number of vibro-rod drivings was 849. The quantity of soil, 6600 m³, was cast from the ground surface into the annular spaces generated between the oscillating vibro-rod and the surrounding in-situ fill. The quantity of the cast soil corresponds to a rate of 1.2 to 1.3 m³/m² over the treated area.

Fig. 3. Vibro-rod used

Fig. 4. Vibro-rod compaction process

Post-Treatment and Post-Earthquake SPT *N*-value Profiles

Figure 5 shows the SPT *N*-value profiles before and after the treatment works. Solid and open circles denote the minimum and average *N*-values after the compaction over the treated site. It can be seen in the figure that the *N*-values of the reclaimed fill increased considerably due to the vibro-rod compaction works.

Figures 2(b) and 6 show the *N*-value profiles obtained at three points, Nos. a-1 to a-3 shown in Fig. 1, in 1997, two years after the quake. The fines content of the fill and the average post-treatment *N*-value profile are also shown in the figure. The post-earthquake *N*-value profiles almost agree with the post-treatment ones in a broad sense, except for *N*-values higher than 40 in gravel layers.

Outline of Buildings and Piles Investigated

Building A and Building B were constructed at the treated site with another building as shown in Fig. 1. Building A is a four-story steel structure 24 m wide in the span direction and 20 m long in the ridge direction, as shown in Photo. 2. The building is supported by a steel pipe pile foundation. In contrast, Building B is a one-story steel structure with a spread foundation, as also shown in Photo. 2 (the right-hand neighbor).

Fuller piles about 40 m long with closed ends were used. The outside diameter of the upper and middle piles is 500 mm, while the lower, 10 m-long piles have an outside diameter of 600 mm. All the piles have a 12 mm wall thickness. On the outside surface of the upper and the middle piles, asphalt slip-layers 6 mm thick were applied over a length of 24 m in order to reduce negative skin friction caused by ground settlement due to the consolidation of the Ma13 layer (Tanahashi et al., 1987). The piles were driven with an M43-type diesel hammer through bore-holes pre-drilled with an earth auger in the compacted fill deposits. The pre-drilled holes were 15 m deep. The

Fig. 5. Pre- and post-treatment *N*-value profiles

Fig. 6. *N*-value and fines content profiles after earthquake

Photo. 2. Building A and Building B

Fig. 7. Settlement time histories of the two buildings

piles have penetrated through the fill, the Ma13 clay layer and the underlying sandy soil layers to a bearing stratum.

ESTIMATION OF GROUND SUBSIDENCE DUE TO EARTHQUAKE

The settlement of Buildings A and B had been regularly measured since 1981. Figure 7 shows the last part of the time histories of the settlement of the two buildings. The measurement ceased in August 1990. In March 1997, two years after the quake, the settlement was measured again. The settlement values without any effect due to the

quake can be extrapolated using a hyperbolic relationship that is considered proper. Comparing the estimated settlement with the measured one, we can get the settlement due to the earthquake concerning the two buildings. As Building B has a spread foundation, the settlement of Building B can be regarded as ground subsidence. Consequently, from Fig. 7, the ground subsidence due to the earthquake is estimated to be 30 to 100 mm. This value is much smaller than that measured at other apparently liquefied but untreated sites on Port Island. Thus, the effect of treatment can be recognized concerning the ground subsidence.

LIQUEFACTION RESISTANCE OF COMPACTED FILL AFTER EARTHQUAKE

In order to evaluate the liquefaction resistance of the compacted fill, high-quality undisturbed samples were obtained at the point shown in Fig. 1 using the in-situ freezing method in the same manner as described by Hatanaka et al. (Hatanaka et al., 1997). The frozen samples were taken from depths of 7 to 18 m below the ground surface. Fragments of concrete and asphalt could be seen in some samples, as well as rocks and silt blocks.

Liquefaction tests were performed with a triaxial apparatus for cylindrical specimens 150 mm in diameter and 300 mm high. The specimens were thawed and saturated with the help of CO₂ gas and back pressure, until the pore pressure coefficient B -value reached 0.96 or more. Each specimen was then consolidated isotropically under a confining stress that equaled the vertical effective stress of 127 to 206 kPa in situ. After the specimen had been consolidated for more than 1 hour, the test was performed by applying sinusoidal loading at a frequency of 0.02 Hz.

Physical properties of the twelve specimens are as follows: Fines Content, F.C.=1.2 to 4.0%, Gravel Content, G.C.=42 to 81%, D_{50} =1.4 to 51.4 mm, U_c =1.5 to 148. Normalized SPT N -values at a vertical effective stress of 98 kPa, N_1 , at the depth where the samples were taken, range from 10 to 20 with the exception of the large N -values resulting from the striking of the spoon sampler against the fragments of rocks etc..

Figure 8 shows the relationship between the applied cyclic shear stress ratio and the number of load cycles required to cause a double amplitude axial strain of 5% (hatched range), and to cause an excess pore water pressure ratio of 95% for the specimens obtained from the compacted fill. If the liquefaction resistance is defined as the value of the cyclic stress ratio required to cause a double amplitude axial strain of 5% in a number of cycles of 15, the liquefaction resistance of the compacted fill becomes 0.27 to 0.35, except for three specimens having

much greater resistance to liquefaction. Figure 8 also shows the results on the specimens obtained from the high-quality undisturbed samples taken from the untreated reclaimed fill deposits on Port Island, which were reported by Hatanaka et al. (Hatanaka et al., 1997). The liquefaction resistance is 0.15 to 0.22 for the untreated fill. Consequently, the compacted fill has 60 to 80 percent greater resistance to liquefaction than the untreated fill, if direct comparison is allowed. However, the compacted fill was judged to have liquefied completely despite having some layers that did not undergo liquefaction, according to the present method described in the "Recommendations for Design of Building Foundations" (AIJ, 1988). The layers that did not undergo liquefaction are also considered to be affected significantly not only by the upward flow of water as pointed out by Yoshimi et al. (Yoshimi et al., 1975), but also by the lateral flow of water from the surrounding liquefied fill deposits that had not been treated.

INVESTIGATIONS OF DEGREE OF INTEGRITY FOR FOUNDATIONS AND SUPERSTRUCTURES

Measurement of Differential Settlement and Tilt of Buildings

For Building A, measurements of differential settlement and tilt of the superstructures, and excavation observation for the foundation were conducted together with television observation and a pile inclinometer survey for the piles. The differential settlement and the tilting were also measured for the superstructure of Building B.

Figure 9 shows the differential settlement of the two buildings, (a) indicating the settlement values relative to each datum point of the buildings in February 1990, before the quake, and (b) indicating the values two years after the quake. The differential settlement of Building B considerably decreased, while that of Building A was almost retained. Maximum differential settlements of 63 mm and 164 mm were measured for Building A and

Fig. 8. Liquefaction resistance of compacted fill

Fig. 9. Differential settlement of the two buildings

Building B at the end of 1988, respectively (Tanimoto and Fukui, 1990). The differential settlement of Building B might have been reduced by the earthquake or the new reclamation that had been done on the south-side sea.

Figure 10 shows the tilt of the walls of the superstructure at four corners as well as the tilt of footing beams, which was calculated from the settlement values in Fig. 9. Building A tilted by about $1/300$ radians in the north-northwest to northwest direction, having a tilt of about $1/320$ radians and about $1/820$ radians in the two orthogonal directions. It is said that a tilt of less than $1/250$ radians can be borne for many years from a psychological point of view. The measurement of Building A revealed that the building would still be usable with the tilt within the allowable limit.

Excavation Observation for Foundation

The ground was excavated in order to investigate the integrity of pile caps, footing beams and the head parts of the four piles, including Nos. 4 to 6 piles shown in Fig. 11, around the northeast corner of the building. A few hair-line cracks were observed on the bottom surfaces of the pile caps, extending outward from the periphery of the piles. But, no lateral gaps could be seen at the pile-pile cap connections of the four piles. Diagonal cracks were found in the pile cap supported by No. 6 pile. However, severe damage resulting in interference with the structural soundness of the superstructure did not occur in the footing beams and pile caps. The deflection of the footing beam was measured at intervals of 0.5 m between the northeast corner and a point 12 m apart from the corner along the X-direction. The outward deflection of 13 mm was measured as the maximum value. The piles were spirally welded manufactured goods.

Fig. 10. Tilting of the two buildings

Fig. 11. Locations of piles investigated

Television Observation for Piles

Large holes were made in the piles as shown in Photo. 3 to measure the length of the inside of the hollow piles using a plumb line. The television observation was conducted inserting a remote-controllable television camera (Oh-Oka et al., 1997a) into the air-filled inside of the piles through these holes. The objective of the television observation was to confirm the position of the pile joints that had been welded in the field, and the extent of damage both to the piles themselves and to the field joints of the piles. The observation was ended at a maximum depth of 26.5 m below the bottom of the pile cap for No. 6 pile, because of the existence of soils that had been thrown into the inside of the hollow pile during construction works. The position of the upper field joints was detected at depths of 12.87 m, 12.75 m and 12.50 m below the bottoms of the pile caps for the No. 4, No. 5 and No. 6 piles, respectively. Visible damage could not be found on the inside walls of the piles along the entire length surveyed, including a position with the maximum curvature of pile deformation, which was detected by the pile inclinometer survey.

Photo. 3. Steel pipe piles exposed

Pile Inclinator Survey

The piles were split to a length of about 3 m below the bottoms of the pile caps in order to arrange for the pile inclinometer survey. Some halves were removed, while the remaining halves were left in situ, supporting Building A as shown in Photo. 4, with the help of temporary strong steel structures. The hollow piles were confirmed to be strengthened with reinforced concrete 1 m long in the inside below the pile-pile cap connections, as shown in Fig. 12. The details of this connection lead one to presume that the piles performed under the fixed head condition during and after the quake.

The pile inclinometer survey was conducted for piles Nos. 4 to 6, using the building, while for piles Nos. 1 to 3 and No. 7, it was done through cored holes, 150 mm in diameter, in the reinforced concrete cast below the pile-pile cap connections, after the building had been demolished. Two kinds of pile inclinometer were used, SC-type and BTL-type inclinometers. The details of measuring instruments and procedure have been described elsewhere (e.g., Oh-Oka et al., 1997a). The SC-type and the BTL-type pile inclinometers used are shown in Photos. 5 and 6, respectively. They were inserted into the inside of the hollow piles after the inside was filled with water.

Figure 13 shows typical examples of the deformation mode, i.e., the deformation mode of pile Nos. 4 to 6 in

Photo. 5. SC-type pile inclinometer

Photo. 6. BTL-type pile inclinometer

Photo. 4. Pile inclinometer survey

Fig. 12. Details of pile-pile cap connection

the X-direction, which were obtained with the two kinds of pile inclinometer. Close agreement between the deformation modes evaluated by the different types of pile inclinometer indicates that the pile inclinometer survey is an effective means for obtaining the correct deformation mode of piles, i.e., the permanent horizontal displacement distribution of the piles along the depth after the earthquake.

Figure 14 also shows the typical deformation modes of the piles, i.e., those of piles Nos. 1, 2 and 7 both in the X-direction and in the Y-direction, respectively. They were

Fig. 13. Typical examples of obtained deformation mode of piles (Nos. 4 to 6 piles)

Fig. 14. Typical examples of obtained deformation mode of piles. (Nos. 1, 2 and 7 piles)

obtained using the BTL-type pile inclinometer. Piles Nos. 1 to 6 had been embedded by about 0.1 m into pile caps that had been connected with the same reinforced concrete beam. Only pile No. 7 was located on the opposite side of the building. The ordinate of the figures denotes the depth represented with Kobe Point denotation. Shaded parts in the figures indicate the Ma13 layer.

An Interpretation on Permanent Pile Deformation due to Earthquake

The permanent deformation mode due to the earthquake can be estimated by subtracting that before the quake from that detected by the pile inclinometer survey after the quake. The initial permanent deformation mode before the quake usually means the permanent deformation mode just after the completion of the installation of all the piles. However, in the case of Building A, all the piles had been driven 4 to 5 months after the completion of the reclamation works at the wharf surrounded by the sea on three sides. Therefore, lateral deformation of the reclaimed fill and the Ma13 layer themselves, which had been induced by the differential consolidation settlement of the Ma13 layer itself, might have caused the deformation of the piles by the occurrence time of the earthquake, as Matsui et al. have pointed out (Matsui et al., 1988).

The following have to be considered in order to estimate the initial permanent deformation mode of the piles.

- (1) The piles are not necessarily installed in a perfectly vertical direction.
- (2) The piles are sometimes driven into soil deposits changing the direction gradually or abruptly when their tips strike against something such as cobbles or boulders.
- (3) The piles are not necessarily jointed straight by the field weld.
- (4) The reclaimed fill on the soft clay layer continues to suffer lateral deformation caused by the differential consolidation settlement of the clay layer itself. This phenomenon continues to occur especially around the quay wall for a long term after the completion of the reclamation works, leading to deformation of the piles in the reclaimed fill and soft clay layer before the earthquake.

Therefore, it is not so easy to estimate the permanent deformation mode of piles caused by the quake, if few piles are surveyed only up to limited depths. However, the following should also be pointed out:

- (5) Some results obtained by the pile inclinometer surveys, which were conducted at locations apart from the wharf, indicate that the piles were deformed only in the reclaimed fill deposits that were evaluated as liquefied by the earthquake (e.g., Satake et al., 1997).
- (6) The alluvial clay layer (Ma13 layer) with an N -value of 4 to 6 is considerably stiff, having almost the same elastic modulus as reclaimed gravelly fill deposits with a N -value of about 20 under a no pore water pressure build-up condition. Therefore, the piles are hardly considered to be deformed in the Ma13 layer.

(7) The earthquake-induced horizontal displacement vectors of all the piles that are connected with the same footing beam, coincide with each other at the pile heads, provided that the footing beams suffer no damage, that no lateral gaps occur at the pile-pile cap connections, that the piles are point bearing piles, and that the building itself is not rotated on a horizontal plane.

(8) Ground deformation caused by the differential consolidation settlement of the alluvial clay layer has a tendency whereby the maximum curvature occurs around the middle level of the clay layer itself. There is also a tendency that the overlying reclaimed fill is deformed landward in an almost straight line, while the clay layer itself being deformed seaward making a curve shaped like the Japanese character, “<” (Matsui et al., 1988). It can be seen from Fig. 14 that the tendency in the Y-direction agrees with this phenomenon, while, that in the X-direction is rather contrary to it. Therefore, hereafter, the above-mentioned effect pointed out by Matsui et al., as is seen somewhat in the Ma13 layer, will, with one exception, be ignored in the following considerations. The deformation mode of the piles caused by the quake, is calculated under this assumption. However, given is the notice that the calculated deformation mode due to the quake is hardly affected by this assumption above the upper interface of the Ma13 layer, while that is influenced considerably beneath the lower interface of the Ma13 layer if the assumption is not good, to lead to the overestimation of the horizontal displacements of the piles.

Considering the above-mentioned eight matters as well as the pre-drilling and driving records of the piles, the horizontal pile displacement vectors due to the quake were evaluated and were divided into two components as explained below.

- (A) Based on (5), (6) and (8) above, two supplement lines are drawn upward and downward starting from the point showing the maximum curvature around the middle level of the Ma13 layer, as shown with dotted lines in Fig. 14, as the initial deformation mode of each pile.
- (B) The supplement lines are normally straight, though sometimes curved lines are used, considering (2) and (3) above.
- (C) The deformation modes due to the quake are estimated by subtracting the initial deformation modes from those detected by the pile inclinometer survey.
- (D) Finally, calculated horizontal pile displacements in the X- and Y-directions are corrected based on (7) above.
- (E) Consequently, the horizontal pile displacements due to the quake are divided into two components. One component is the pile head displacement vectors above the interface between the reclaimed fill and the underlying Ma13 layer, as shown in Fig. 15(a). The other one is the horizontal pile displacement vectors relative to the fixed pile tips at the level of the lower interface of the Ma13 layer, as shown in Fig. 15(b).

In the Hyogoken-Nambu earthquake, Kobe City succeeded in obtaining the valuable instrumental records of the pore water pressure build-up in the alluvial sandy soil layer underneath the Ma13 layer with the monitoring sys-

Fig. 15. Pile displacement vectors: (a) above the upper interface of Ma13 layer, (b) below the lower interface of Ma13 layer

tem registering at intervals of six hours on Port Island (Phase II) (Kobe City, 1995). According to these records, the excess pore water pressure ratio came to equal 0.5, thirteen minutes after the quake, to lead to 0.6 of the induced maximum value of the excess pore water pressure ratio immediately after the quake (Hiraoka et al., 1996). Recently, it was pointed out that the upper part of the Pleistocene sand layer (diluvial sand layer) of the Tokyo formation had been liquefied in a past earthquake in which the seismic intensity VI had been estimated at the site (Mori and Ikeda, 1997). Consequently, one can assume that the alluvial or diluvial sandy soil deposits overlaid by the Ma13 layer might have been liquefied or at least be in an almost liquefied state during the earthquake on Port Island. In this sense, the result of the simulation analysis made by Fuchimoto et al. is noteworthy, especially concerning the result shown in Fig. 16 (Fuchimoto et al., 1996). They analyzed the problem on a two-story pile-supported steel structure located 100 m from the quay wall. PHC piles, 400 to 500 mm in diameter and 42 m long, had been installed penetrating through an un-

treated reclaimed fill 15 m thick, an Ma13 layer 13 m thick and an alluvial sandy soil layer 6 m thick to a diluvial bearing stratum. In order to estimate the permanent deformation mode of the piles, a static total stress analysis was made for a two-dimensional model, subjected only to body force, and using values of post-liquefaction ground stiffness, which were reduced in accordance with the values of the excess pore water pressure built up during the simulation earthquake. Figure 16 shows the calculated permanent deformation mode of the piles, which was caused only by the post-liquefaction lateral spreading. Fuchimoto et al. concluded that the piles were deformed not only above the upper interface of the Ma13 layer, but also beneath the lower interface of the Ma13 layer, by the effect of the post-liquefaction lateral spreading.

Figure 17 shows the permanent deformation mode of piles Nos. 1, 2 and 7, which is judged to have been caused by the earthquake on the basis of the above-mentioned considerations. It is concluded that all the pile heads of Building A moved by at most 344 mm in the north-north-east to northeast direction, assuming that no lateral shift occurred at the pile tips. The mean value of the pile head displacements due to the pile deformation caused above the interface between the compacted fill and the Ma13 layer is 274 mm (227 mm in the X-direction, and 154 mm in the Y-direction on the average), while the mean value of the remainders due to the pile deformation caused in the sandy soil layers is 81 mm (29 mm in the X-direction, and 76 mm in the Y-direction on the average).

Fig. 16. Calculated example of pile deformation due to post-liquefaction lateral ground spreading (Fuchimoto et al., 1996)

DISCUSSION

Empirical Relationship between D_B/H and X/D_0^2

Ishihara et al. presented the empirical relationship between the ratio of D to D_0 and the ratio of X to L (Ishihara et al., 1997), in which D and D_0 denote the lateral displacement of the ground surface and the lateral displacement of the quay wall crest, respectively. X and L are the distance from a waterfront and the maximum distance affected by the lateral spreading, respectively.

Fig. 17. Permanent deformation mode of piles, judged as caused by the quake

Yasuda et al. (1997) presented another empirical relationship between L and the ratio of D_0 to N_1 . N_1 denotes the average normalized SPT N -value before the quake in liquefied and flowed layers, which is proposed by Yasuda et al.. According to the relationship, L seems proportional to D_0/N_1 , although the data scatters over a wide range. Substituting $L = \alpha D_0/N_1$ into the relationship between D/D_0 and X/L , one can get Fig. 18, where, α is a constant on the extent of influence due to the quay wall shift.

On the other hand, other data (e.g., Kobe City, 1995, Tokimatsu et al., 1997a) show that the displacements of the ground surface are decreasingly distributed from the waterfront extending far back with a certain residual quantity in inland areas. The residual value, ΔD , is judged to be about $0.1D_0$ according to the above-mentioned data. Therefore, in Fig. 18, the increment of the ordinate $\Delta D = 0.1D_0$ has to be added to the original values of the ordinate.

In order to simplify the relationship and to find the significant factors, the hyperbolic relationship represented by the solid line will be hereafter adopted instead of the original broken line proposed by Ishihara et al., although the solid curve does not agree with the broken curve and the values of D/D_0 exceed 1.0 when the value of X/L becomes less than 0.2. The disagreement will be modified empirically using data obtained by the pile inclinometer survey (Figs. 20, 22 and 23). The hyperbolic relation is given by Eq. (1).

$$D \propto \alpha \cdot \frac{1}{N_1} \cdot \frac{D_0^2}{X} \quad (1)$$

Tokimatsu et al. presented theoretical solutions on the deformation mode of piles subjected to ground

Fig. 18. Assumed relationship between D/D_0 and X/L

deformation due to liquefaction-induced lateral spreading on the basis of the theory about an elastic beam supported by elastic media (Tokimatsu et al., 1997b). The permanent displacement of the pile heads, i.e., the permanent moved distance of the building, D_B , can be calculated using D , β , H under some given conditions. D denotes the lateral displacement of the ground surface beneath the building, β is a pile-soil characteristics parameter given by Eq. (2) and H is the thickness of the liquefied soil deposits.

$$\beta = 4 \sqrt{\frac{KhB}{4EI}} \quad (2)$$

in which Kh and B denote the coefficient of horizontal subgrade reaction and the outside diameter of the pile, respectively. E and I denote the elastic modulus of a pile material and the moment of inertia of the cross-section

Fig. 19. Used relationship between D_B/D and βH

of the pile, respectively. Figure 19 shows the relationship among the above-mentioned four parameters, which is calculated under the assumption that the piles are deformed with a fixed head condition by the effect only of the lateral deformation of the liquefied soil deposits. The distribution of the horizontal displacement of the ground along the depth is given by a cosine function. It can be seen in the figure that $D_B/D \doteq 1$ when the values of βH exceed about 3.3, and that $0 \leq D_B/D < 1$ when the values of βH are less than 3.3.

Tamura et al. reported that the wide range of $0 \leq D_B/D < 1$ was obtained by field investigation and aerial photograph survey for pile foundations of the piers supporting highway bridges at liquefied sites (Tamura et al., 1997). In general, the drilled shafts (cast in place concrete piles) are, if anything, apt to have less than 3.3 as the values of βH , while the precast piles have a tendency to have more than 3.3, under similar conditions apart from the kind of piles. The straight broken line in the figure is adopted in this study to simplify the numerical expressions. Then, the relation can be expressed as follows:

$$\frac{D_B}{D} \propto \beta H \quad (\beta H \leq 3.3) \quad (3)$$

By substituting Eq. (1) into Eq. (3), one can get the following equation.

$$\frac{D_B}{H} \propto \alpha \cdot \beta \cdot \frac{1}{N_1} \cdot \frac{D_0^2}{X} \quad (\beta H \leq 3.3) \quad (4)$$

If βH exceeds 3.3, the relation can be expressed as follows:

$$\frac{D_B}{D} \doteq 1 \quad (\beta H > 3.3) \quad (5)$$

Then,

$$\frac{D_B}{H} \propto \alpha \cdot \frac{1}{H} \cdot \frac{1}{N_1} \cdot \frac{D_0^2}{X} \quad (\beta H > 3.3) \quad (6)$$

Application of Empirical Relationship to Buildings Adjacent to Quay Walls

The Building Research Institute has compiled useful data from various sources and has, as well, produced valuable data by means of television observation and the pile inclinometer survey. Some of the data related to liquefaction are tabulated in Table 1 (Oh-Oka and Tachikawa, 1997b).

Picking out the pile-supported buildings adjacent to the quay walls from Table 1, and, for the time being, passing over the differences in α , β , N_1 and H , we can get a simple relationship shown in Fig. 20, where H is not the thickness of the liquefied layer but that of the reclaimed fill. The buildings are not necessarily moved in a direction perpendicular to the waterfront line. On that account, in the figure, D_B^* and X^* are used as the components of D_B and X in a direction perpendicular to the waterfront line, respectively, i.e., D_B^* denotes the moved distance of the building in a direction perpendicular to the shore line. X^* denotes the distance between the building and the quay wall in a direction perpendicular to the shore line. It seems that there exists a strong correlation between D_B^*/H and X^*/D_0^2 as shown in the figure which only uses comparable cases located on the mainland of Kobe City and Ashiya City with concrete piles having a diameter of 400 to 500 mm.

Characteristics of Special Condition of Location, "Wharf"

The special condition of the location of Building A, i.e., Building A being located at the wharf, influences the deformation mode of the piles subjected to ground deformation induced by the quake in the following two particular points.

(1) The Ma13 layer is cut off completely on three sides through being replaced with sand fill (usually called replaced sand) with an inverted trapezoid-shaped cross section underneath the continuous quay walls. Therefore, the Ma13 layer 12 m thick is considered to be easier to move laterally as compared with that under other conditions, if excess pore water pressure is built up in the replaced sand as well as in the sandy soil layers overlaid by the Ma13 layer.

(2) The two quay walls run parallel to each other. The effect of the driving forces induced by the outward lateral shift of the two compensates each other, in accordance with the distances between the building and the quay walls.

Consideration on Lateral Shift of Building A

Figure 21 shows the condition of the location of Building A and the lateral displacements of the quay wall crests located nearby, including the lateral displacement of the ground surface. The displacements are estimated from various sources (e.g., Hamada et al., 1995; Inagaki et al., 1996; The Port and Harbour Research Institute, 1997). A US-Japan geotechnical reconnaissance team visited this site and stated that "The sea wall across the street displaced to the south a distance of 20 to 30 cm," on the basis of the observed total width of the openings in the ground (US-Japan geotechnical reconnaissance

Table 1. Compiled data by the Building Research Institute

District	Building investigated	Story of building	Type of pile	Diameter of pile (mm)	Thickness of fill (m)	Condition of location (Number of quay walls)	Investigation method used
Port Island and its vicinity	DAI	7	Pedestal	430	5.5	Adjacent to quay wall (1)	Inclinometer (SC)
		2	PHC	450			
	KIT	2	PHC (A-type)	450	17	Inland	Inclinometers (BTL + SC)
	NIS	2	PHC (A-type)	500	17	Inland	Aerial photo survey
	PIZ	4	SC + PHC(B-type)	500	18	Inland	Aerial photo survey
	TAI	1	PC(A-type)	400	20	Wharf (3)	Inclinometer (SC)
Fukaehama and its vicinity	12B	4	Steel Pipe	500 + 600	20.5	Wharf (3)	Inclinometers (BTL + SC)
	KOB	3	PC(A-type)	400	8	Adjacent to quay walls (2)	Inclinometer (SC)
	SHI	3	PHC (B-type) + PHC(A-type)	450	15.5	Adjacent to quay wall (1)	Measurement + Aerial photo
	TOB	2	Steel Pipe	406.4	15	Relatively adjacent to quay walls (2)	Inclinometer (BTL)
	HIG	4	PHC (A-type)	350	13	Inland	Inclinometer (SC)
	MEI	3	PC (A-type)	450	9	Inland	Inclinometer (BTL)
	TOW	11	PC(A-type)	500	8	Adjacent to quay walls (3)	Inclinometer (SC)

Fig. 20. Empirical relationship between D_B^*/H and X^*/D_0^2

team, 1995). However, in this figure, the displacement of ground surface measured based on the aerial photograph survey by Hamada et al. is used.

As shown in Photo. 1 taken shortly after the quake, the south-side sea was partitioned off and reclamation works were being done when the earthquake took place. For that reason, the bottom of the south-side sea was shallower than the other two. The lighter color of the partitioned sea as shown in Photo. 1 indicates this fact, lead-

Fig. 21. Condition of location of Building A and lateral displacements of quay walls and ground surface

ing to less lateral displacement of the quay wall facing the south-side sea as compared with that of the other two quay walls.

The values of X^*/D_0^2 can be calculated according to the distance of Building A from the quay walls and the lateral displacement of the quay wall crests, which are shown in Fig. 21.

$$X^*/D_0^2 = 16.5 \text{ m}^{-1} \text{ in the positive Y-direction.}$$

$$X^*/D_0^2 = 24.2 \text{ m}^{-1} \text{ in the positive X-direction.}$$

$$X^*/D_0^2 = 61.7 \text{ m}^{-1} \text{ in the negative Y-direction.}$$

Using the empirical relationship shown in Fig. 20, one can get values of D_B^*/H as follows:

$D_B^*/H=3.8\% \rightarrow D_B^*=0.78$ m in the positive Y-direction.

$D_B^*/H=3.0\% \rightarrow D_B^*=0.62$ m in the positive X-direction.

$D_B^*/H=0.6\% \rightarrow D_B^*=0.12$ m in the negative Y-direction.

where the thickness of the reclaimed fill, 20.5 m, is used as the value of H , for Fig. 20 was made by adopting the thickness of the reclaimed fill as that of the liquefied layer, H . Therefore, the components of D_B in the positive X- and positive Y-directions are calculated to be 0.62 m and 0.66 m (0.78 m–0.12 m), respectively. The calculated values are much greater than the measured ones (0.23 m and 0.15 m in the positive X- and positive Y-directions, respectively). However, moved directions coincide qualitatively with each other.

In Fig. 22, the values of D_B/H are plotted against those of X/D_0^2 on a semi-logarithmic scale using all the data tabulated in Table 1, where, D_B and X denote the moved distance of the building and the distance between the building and the quay wall in the moved direction, respectively. Solid and open circles mean the data obtained on Port Island and its vicinity and on Fukaeahama and its vicinity, respectively. Though there is insufficient data, it seems from the figure that the values of D_B/H on the buildings located at the liquefied sites on Port Island are smaller compared with those at the liquefied sites in Fukaeahara district. Accordingly, the value of D_B has to be calculated using the data obtained only in the Port Island district, although the direction moved is not exactly perpendicular to the shore line.

Then,

$D_B/H=3.0\% \rightarrow D_B=0.62$ m in the positive Y-direction.

$D_B/H=2.6\% \rightarrow D_B=0.53$ m in the positive X-direction.

$D_B/H=1.8\% \rightarrow D_B=0.37$ m in the negative Y-direction.

Therefore, the values of D_B in the positive X- and positive Y-directions are obtained as 0.53 m and 0.25 m (0.62

m–0.37 m), respectively. The calculated values are yet larger than the measured ones, keeping the rough agreement between the measured direction and the calculated one. The difference can be considered to be due to the effect of the vibratory compaction.

Reasons for Pile Deformation Reduction

Under the above-mentioned considerations, the differences in α , β , N_1 and H were passed over. While α is an undefinable parameter, β and N_1 are definite parameters.

β is related both to the reduction of the lateral spring parameter of the soil layers, and to the degree of damage to the pile foundation. Figure 23 shows the same data that are plotted in Fig. 22, being classified by the tilt of the superstructures. The fraction in parenthesis denotes the tilt in radians. In general, the smaller the tilt, the smaller the damage to the pile foundation. The critical value of D_B/H is about 2 percent, separating usable buildings from unusable buildings after the quake. The value of D_B becomes smaller because of the smaller value of β if the bending stiffness of the pile foundation, EI , remains an initial value without any hinges yielded by the quake.

N_1 is directly related to the effect of the treatment. The increase of the value of N_1 by the effect of the vibro-rod compaction can be attributed to a decrease in the value of D_B .

One more reason is that the building is located at the wharf. Compared with the building designated as TOB, Building A (designated as 12B in Figs. 22 and 23) has a lower value of D_B/H along with building TAI located also at the wharf on Port Island (TOB: Satake et al., 1997; TAI: Namba et al., 1997). Building TOB was pushed out seaward by the mass of the liquefied fill deposits moving forward in parallel in one direction. The movement pattern of the building is designated as Type A. On the other hand, at the wharf, Building A (Building 12B) was pushed out in a direction dominated by the combination of three-way ground movements that went away from each other. This movement pattern is designated as

Fig. 22. Empirical relationship between D_B/H and X/D_0^2 distinguishing between the two investigated districts

Fig. 23. Empirical relationship between D_B/H and X/D_0^2 distinguishing between usable and unusable buildings

Type B. Another pattern is designated as Type C. Consequently, the effect of the three-way driving forces at the wharf is considered to become smaller as compared with the effect of the one-way driving force at the site adjacent to one or two quay walls. Symbols A, B and C in Fig. 22 denote these movement patterns of the buildings, respectively.

CONCLUSIONS

The following conclusions may be made on the basis of the field investigations for the steel pipe pile foundation supporting a four-story steel structure that survived the 1995 Hyogoken-Nambu earthquake. Subsurface investigations were conducted at the site for the soil deposits, including the reclaimed fill improved before the quake by the vibro-rod compaction method.

- 1) The ground subsidence due to the earthquake was estimated at 30 to 100 mm.
- 2) The compacted fill had 60 to 80 percent greater resistance to liquefaction than the untreated fill.
- 3) The building tilt before the quake was almost exactly retained after the quake, keeping the value of about 1/300 radians.
- 4) Notable damage resulting in interference with the structural soundness of the superstructure did not occur in the foundation.
- 5) Piles about 40 m long were deformed not only above the interface between the reclaimed fill and the underlying alluvial clay layer (Ma13 layer), but also at the lower interface of the Ma13 layer.
- 6) The pile heads are judged to have been moved by at most 344 mm in the north-northeast to northeast direction.
- 7) It is inferred that the smaller permanent deformation of the steel pipe pile foundation was due to the effect of the vibratory compaction, the high integrity of the piles themselves and the special condition of the location

“wharf”, by using the relationship empirically derived from results obtained mainly by means of the pile inclinometer survey and a simple elastic theory.

- 8) The critical value of D_B/H is about 2 percent, separating the usable buildings from the unusable buildings after the quake, if the allowable tilt of the superstructure is less than 1/250 radians (Fig. 23).

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NOTATION

- B = Outside diameter of pile
 D = Lateral displacement of ground surface at point having distance of X from waterfront
 D_B = Moved distance of building in the moved direction
 D_B^* = Moved distance of building in the direction perpendicular to waterfront line
 D_0 = Lateral displacement of quay wall crest
 E = Elastic modulus of pile material
 EI = Bending stiffness of pile
 $F.C.$ = Fines content
 $G.C.$ = Gravel content
 H = Thickness of reclaimed fill or Thickness of liquefied layer
 I = Moment of inertia of cross-section of pile
 K_h = Coefficient of horizontal subgrade reaction
 L = Maximum distance affected by lateral spreading

N_C = Number of cycles

N_1 = Normalized SPT N -value at vertical effective stress of 98 kPa

u = Excess pore water pressure

X = Distance between building and quay wall in the moved direction of building

X^* = Distance between building and quay wall in the direction perpendicular to waterfront line

α = Constant on extent of influence due to quay wall shift

β = Pile-soil characteristics parameter, see Eq. (2)

ε_{DA} = double amplitude axial strain

σ_d = deviator stress

σ'_{CO} = initial effective confining stress

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