DYNAMIC BEHAVIOR OF A STEEL SHEET PILE QUAY WALL IN THE FULL-SCALE EXPERIMENT WITH REGARD TO LIQUEFACTION

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SUMMARY

The full-scale test by controlled blast induced liquefaction was conducted on November 13, 2001 at the Port of Tokachi on Hokkaido Island, Japan. The focus of this experiment was on the dynamic behavior of full-scale steel sheet pile quay walls and the difference between those walls corresponding to behaviors of seismic coefficients of 0.15 and 0.00. The test results revealed the progress of the deformation behavior of the full-scale quay wall in the process of liquefaction; the liquefaction in backfill caused the reduction in horizontal bearing capacity of anchor piles and the deformation of quay walls. Therefore, the difference between quay walls with and without seismic design in this test was not large in displacement because the anchor piles became ineffective due to liquefaction.

INTRODUCTION

The 1995 Hyogo-ken Nambu earthquake caused severe soil liquefaction in extensive areas of reclaimed land and seriously damaged many of the port and harbor structures in Kobe. Many steel sheet pile quay walls have also been damaged in large earthquakes. However, it is recognized that the seismic stability of sheet pile quay walls may be larger than gravity type quay walls due to less inertia force. Therefore, a new seismic design procedure is required in regard to performance-based design.

Dynamic soil-structure interaction greatly influences the seismic stability of steel sheet pile quay walls. Their seismic performance is often examined with numerical simulation or model tests. However, the motion of full-scale sheet pile quay walls during earthquakes, which is the most important information for the seismic design concept, is not well known in general. In this experiment we conducted a field vibration test by controlled blasting focusing on the dynamic behavior of full-scale quay walls during a liquefied ground state.

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SITE, TEST QUAY WALLS, AND INSTRUMENTATION

This experiment was carried out as part of the "Seismic performance of urban, reclaimed and port areas - full scale experiment at Tokachi port" program in which the Japanese and U.S. research organizations participated centering on the Port and Airport Research Institute, Japan[1]. Two sections of steel sheet pile quay wall with a 5m water depth (seismic coefficient $k_h = 0$: quay wall without seismic design, $k_h = 0.15$: quay wall with seismic design) were built at wharf No. 4 at Tokachi Port, as shown in Figs. 1 and 2. The explosive were arranged three dimensionally in backfill behind the quay, in order to observe the stability of a steel sheet pile quay wall with liquefaction induced by blasting. Other structures for this project, such as pile group foundations, were installed in the backland of the quays as shown in Fig. 3.

Standard penetration test results and particle size distribution of the soil obtained from the area where the quay walls were constructed are shown in Figs. 4 and 5, respectively. The layer deeper than -5m is gravely sand whose N-value is about 40. The layer shallower than -5m is reclaimed material whose N-value is from 0 to about 5. The layer shallower than 1m is comprised of sandy material which is easy to liquefy, as can be seen in Fig. 5.

The arrangement of measuring devices and emulsion explosives is also shown in Fig. 1. The sectional views of B-B and E-E are shown in Fig. 6. Seismic and non-seismic designed quays are shown in Fig. 7; SP-IIIw type piles (SY295) and H-section steels (H350x350x12x19, SS400) were used for sheet piles and anchored piles, respectively. Steel bars of 46mm in diameter (SS400) were used for tie-rods. The quay walls with and without seismic design were different from each other in the number of anchor piles and
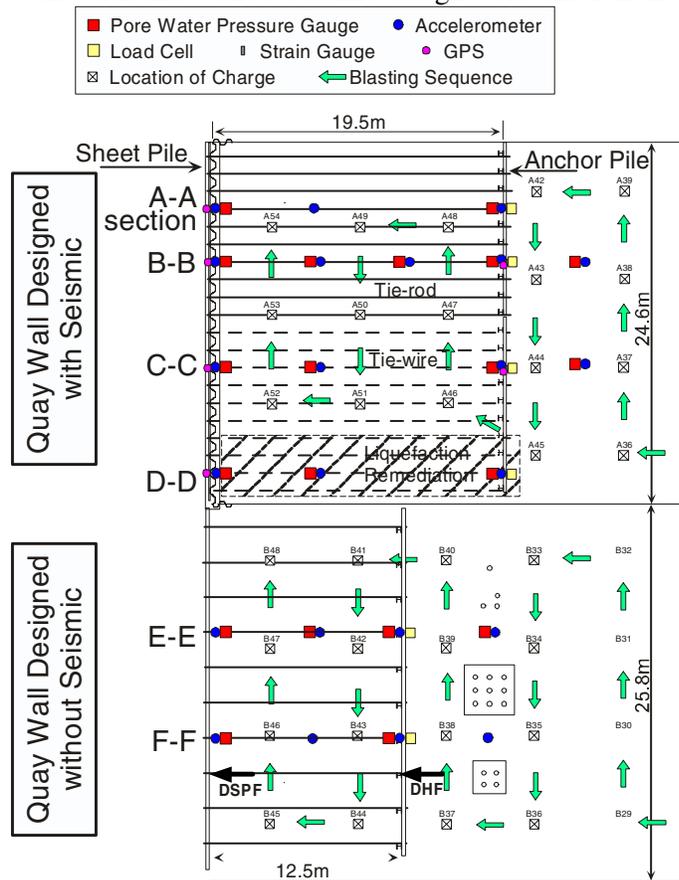

Figure 1 Floor plan of test field

Figure 2 Aerial photograph of test field

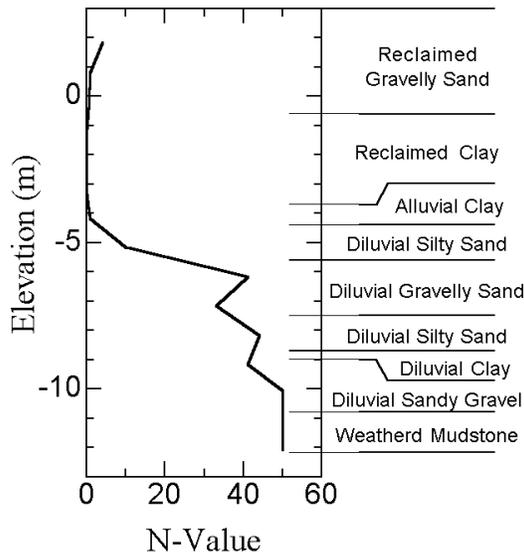Figure 3 Bird's-view photograph of test field
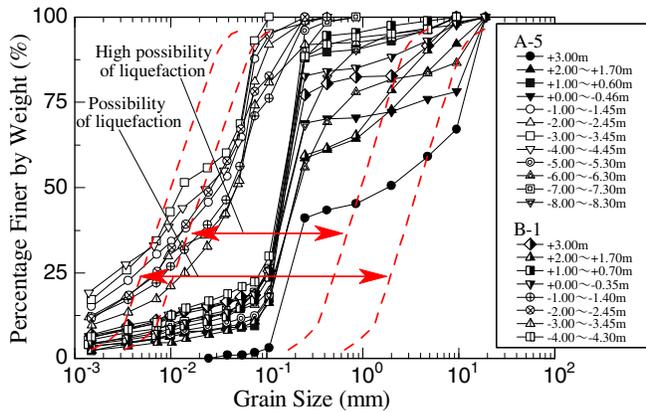

Figure 4 N-value and soil type profile

Figure 5 Grain size distribution

tie-rods, the length of sheet and anchor piles, and the intervals of tie-rods and anchor piles. Moreover, tie-wires and permeable grouting method as liquefaction remediation[2] were employed at a part of the quay with seismic design. Although the quay walls were designed with a water depth of -5m, the water level in front of quay walls was set to +2m (-7m in water depth) at the time of the blasting experiment. The raise in water level was performed to increase saturation in the ground and not to destroy the quays before the blasting test, decreasing total earth pressure acting on the sheet pile. Measurement was performed in the A-A – D-D section in the seismic designed quay and the E-E – F-F section in the non-seismic designed quay, respectively; with strain distortion of a steel sheet piles and anchor piles, acceleration, pore water pressure, displacement (dynamic GPS measurement), etc. Strain distortion of piles and tension of tie-rods were also measured during the construction processes of as tensing tie-rods up after pile installation and excavation in front of the sheet piles.

As shown in Fig. 1, there were many blasting holes at intervals of about 5m. Two explosives causing vibration translation were installed in each blasting hole and blasting was performed one by one at intervals of 0.7sec from the landward side towards the quay wall side. Blasting duration times were 37.3sec and 33.2sec in the quays with and without seismic design, respectively. After the blasting ended in the observation
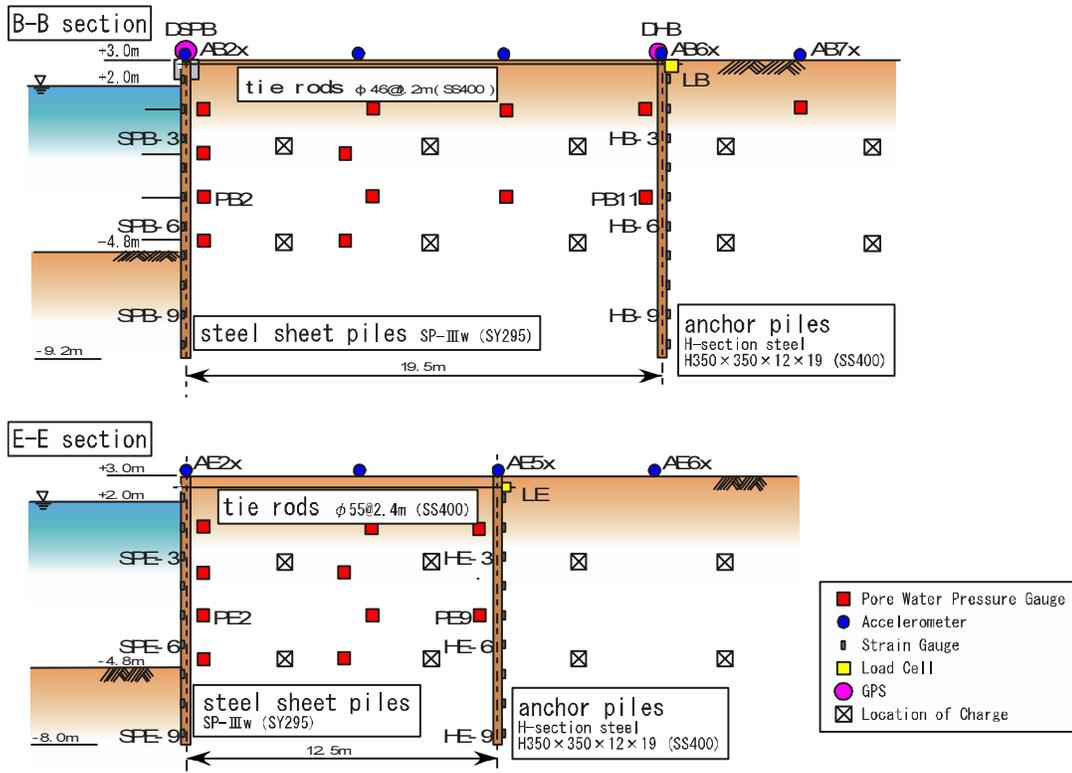
Figure 6 Cross Section of measuring devices arrangement

area where model structures and instruments were installed, continued blasting in the perimeter part of the observation area was performed in order to extend vibration duration time; the whole duration time was set to 62.3sec. Additionally, tie rods of the non-seismic designed quay were cut by forming explosives about 86sec after the blasting start, in order to urge lateral flow of the inclined landfill made behind the quay. However, the tie-rods cutting and the perimeter part blasting are omitted for time history data for this paper; tie-rod cutting was unrelated with the behavior of and vibration in quay walls as the perimeter part blasting was not large.

TEST RESULTS AND DISCUSSION

Behavior during construction

Figure 8 shows the bending strains time histories for the sheet and anchor piles of the seismic designed quay during construction. The test quay walls were the excavation type as illustrated in Fig. 9; sheet and anchor piles were installed into the ground, they were connected with tie-rods, and the ground in front of the sheet piles was excavated. The bending strains largely varied not at the time of connection and tensing up of tie-rods but during the excavation in front of sheet piles. Figure 10 shows the bending moment distribution in sheet piles and anchor piles during construction, obtained from multiplying bending strain by section modulus and Young's modulus. This figure also clarifies that the bending moment varied considerably
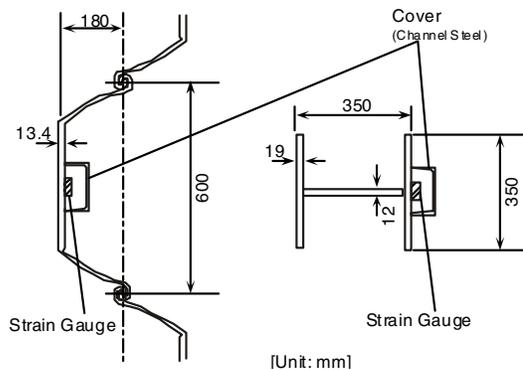

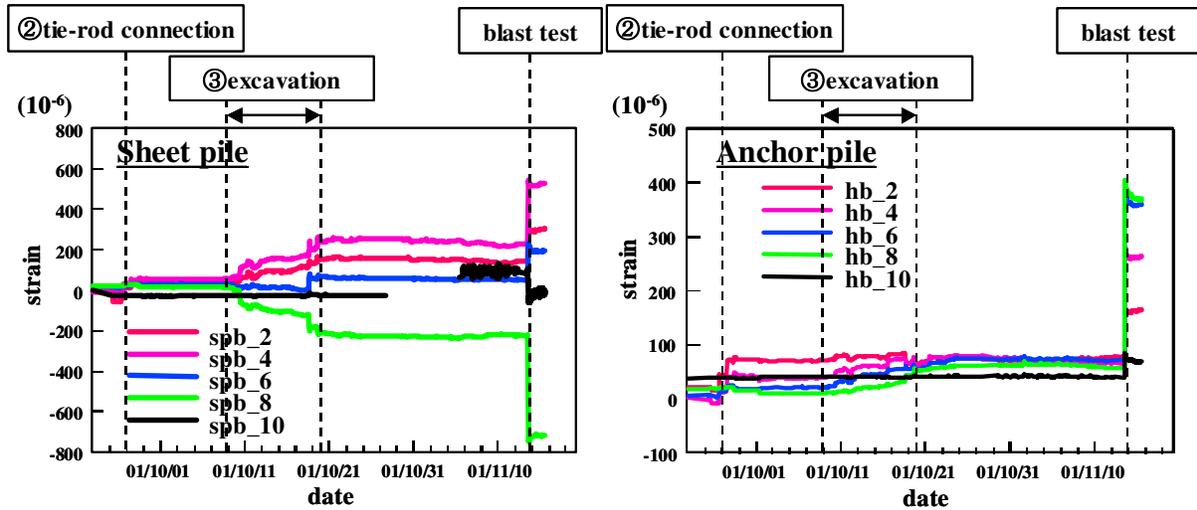Figure 7 Cross section of sheet pile and anchor H-pile

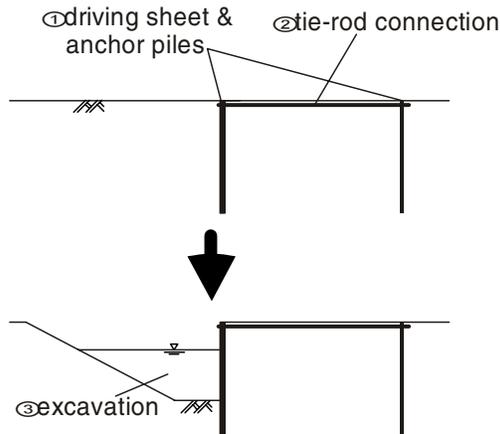Figure 8 Bending strain time histories during construction
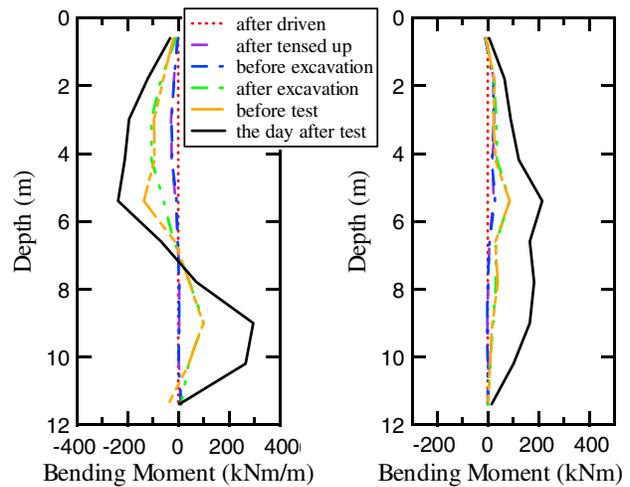

Figure 9 Construction process

Figure 10 Distribution of bending moment

only after the excavation in front of the sheet piles.

Dynamic behavior during blasting shake

Acceleration, displacement and pore water pressure

Figure 11 shows the quay wall before and after the blasting shake. The face lines of both seismic and non-seismic designed quays deformed in a dogleg shape and sand began to boil at the ground surface a dozen minutes after the end of blasting.

Figure 12 shows the time histories for the major measuring points of the B-B (with seismic) and E-E (without seismic) sections. AB2x – AB7x and AE2x – AE6x are horizontal accelerations at the ground surface in the direction vertical to the quay face line in the B-B section of the seismic designed quay and the E-E section of the non-seismic designed quay, respectively. On the basis of these time histories the blasting times of holes A43 behind anchor piles and A53 behind sheet piles in the B-B section were about 36 sec and 46 sec, respectively. The blasting times in the E-E section were a few seconds earlier, and that of blasting holes B39 behind anchor piles and B47 behind sheet piles were about 34sec and 42sec, respec-

tively (see Fig. 4). The maximum horizontal acceleration at the top of the sheet pile for AB2x is about 0.2 – 0.3G when the blasting was advancing at a distance from the quay wall. When blasting advanced to the quay wall side from anchor piles, the acceleration waveform had the pulse peak corresponding to blasting by each explosive. The maximum acceleration at that time was over 10G, and a large acceleration in high frequency arose near the blasting. Since the circumference of AE6x was surrounded with structures, such as pile foundations, it is thought that large acceleration did not arise there.

Figure 11 The quays before and after the test
(a) before the test, (b) after the test

DSPB in Fig. 12 is the horizontal displacement measured with Global positioning system (GPS) at sheet pile top end of section B-B, in the direction vertical to the quay face line. It began to generate from near 35sec when the blasting sequence reached the area just behind the anchor piles (blasting hole A38). Accumulation of displacement then accelerated from near 39sec when the blasting sequence advanced to the sheet piles from the anchor piles. Displacement stopped at the end of blasting and the residual displacement was about 0.42m toward the sea. Displacement at top of anchor pile DHB was almost the same as DSPB, and the steel sheet piles and anchor piles united to behave during blasting. DHF and DSPF indicate displacements in the non-seismic designed quay at the top end of sheet pile and anchor pile respectively, and they also responded together.

Figure 13 shows the residual displacement distribution after the test. At the seismic designed quay, although the displacement at the end of quay face line was a little less than 40cm, the central part was larger than it and the face line deformed in a dogleg shape. The non-seismic designed quay also deformed in a dogleg shape. Residual displacements at DHF and DSPF were smaller than DHB and DSPB because the measurement position of DHF and DSPF was near the ends of the wall face line. The difference of horizontal displacement between the seismic and non-seismic designed quay was not large. However, it should be noted that displacement at the center of the wall face line of the non-seismic designed quay had been larger than that of the seismic designed quay as shown from the video image in Fig. 11.

PB2, PB11, PE2 and PE9 are the instruments used to gather the time histories of pore water pressure in the backfill. The data acquisition after about 38sec at PE9 became impossible due to the nearby blasting shock. Water pressure was gradually increasing during the blasting sequence and had been built up by the nearest blasting; therefore, excess pore water pressure acted on the sheet piles at the same time as the blasting vibration. The excess pore water pressure ratio cannot be calculated accurately because of inarticulacy in water level and a part of ground composition at the instrument installation position. Vertical effective stress at -2.6m and -2.7m were 64kPa and 65kPa, respectively, if the followings is assumed:
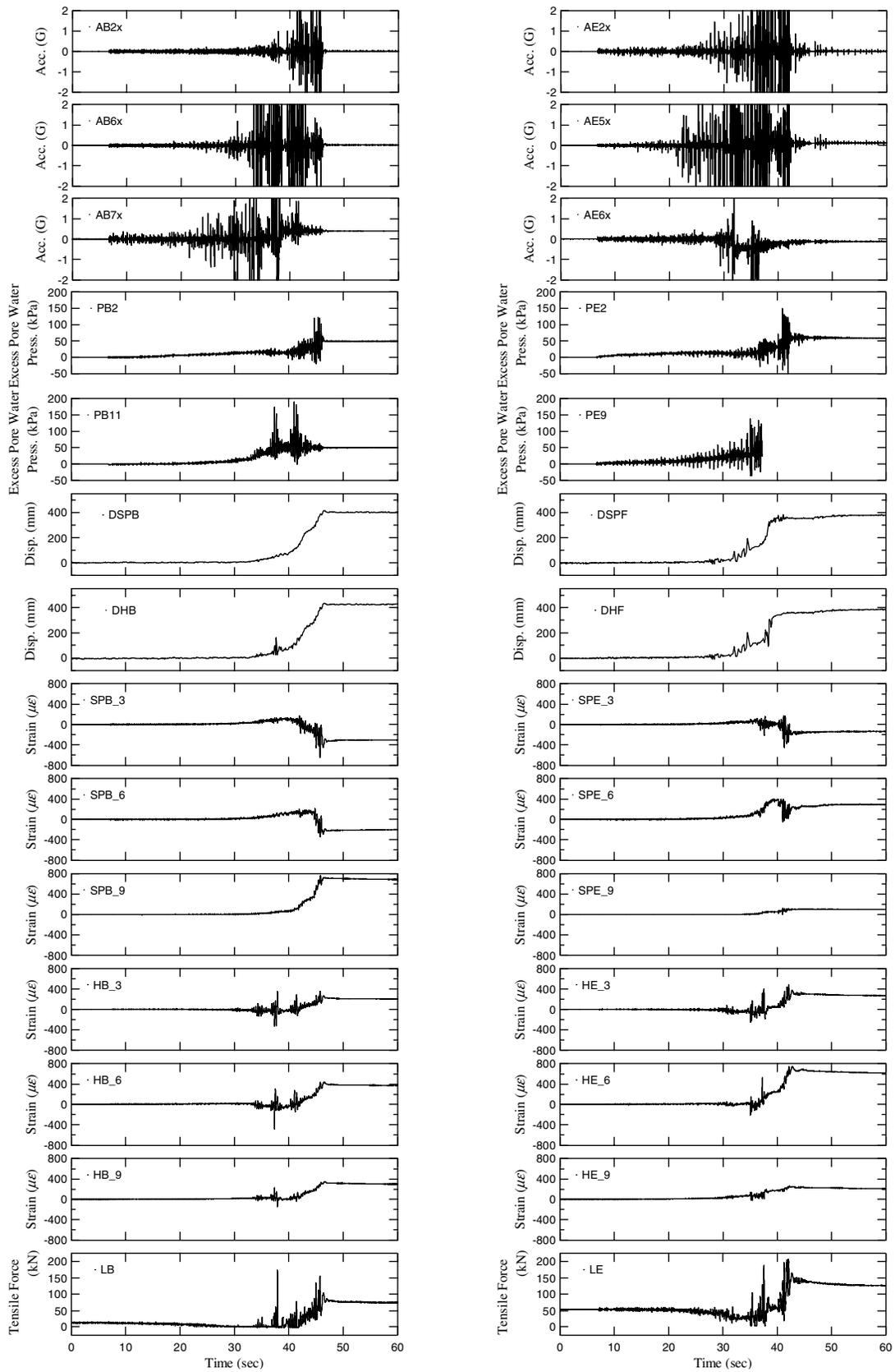
Figure 12 Time histories of acceleration, pore water pressure, displacement, bending strain and tensile force in tie-rod
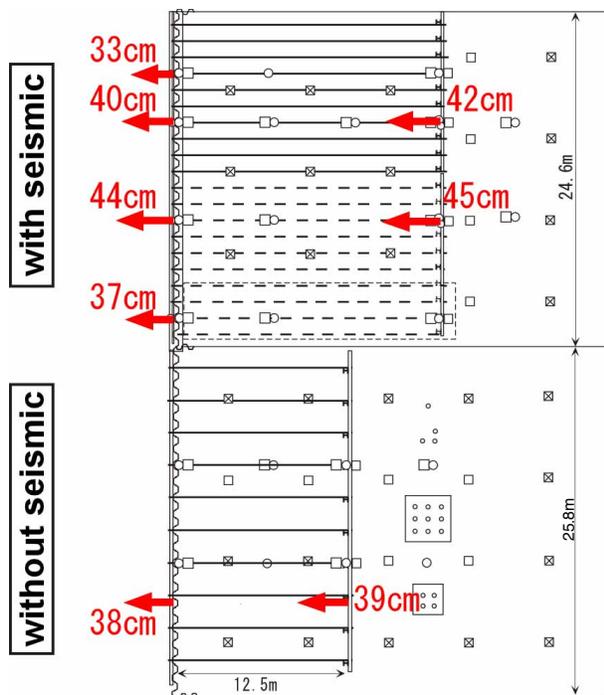
Figure 13 Residual displacement

+2.0m of water level, 10kN/m³ of submerged unit weight under the water level, and 18kN/m³ of unit weight above the water level. Therefore, excess pore water pressure ratio at PB2, PB11 and PE2 were about 0.8, 0.8 and 0.9, respectively, and it is thought that the backfill almost completely liquefied.

Figure 14 shows the long-term time histories of excess pore water pressure at PB4 and PB12. Dissipation of excess pore water pressure took several days. Pore water pressure did not return to the initial value before the blasting and it appears that some water pressure gauges possibly fell as a result of blasting excitation. It is thought that dissipation of excess pore water pressure was completed after 3 or 4 days when the excess water pressure settled down.

As can be seen in Fig. 11, boiled sand phenomenon was not seen near the steel sheet piles but in the backland. Based on excess pore water pressure increasing to almost the initial effective vertical stress (Figs. 12 and 14), the backfill ground appears to have perfectly liquefied during the test. Accordingly, it is thought that re-distribution of excess pore water pressure occurred only in the ground in relation to volume expansion by displacement of the sheet piles toward the sea.

Strain and stress of structural members
SPB-3 – SPB-9 and HB-3 – HB-9 in Fig. 12 indicate the bending strains of steel sheet and anchor piles in section B-B, respectively. SPE-3 – SPE-9 and HE-3 – HE-9 are those in E-E section. Positive strain corresponds to seaward convex deflection of piles and the value before blasting excitation is set to 0.00 in Fig. 12. Regardless of depth, the bending stains of anchor piles increased gradually from near 35sec when blasting sequence reached the anchor piles (blasting hole A38). The deflection in the deep portions of the steel sheet piles, such as SPB-9 and SPE-9, also increased gradually and became convex in
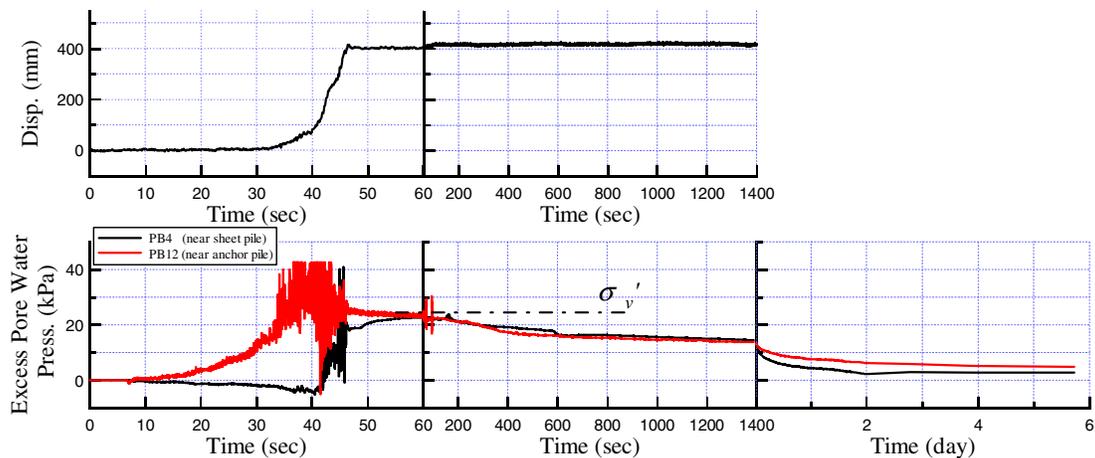

Figure 14 Time histories of displacement and pore water pressure in long term

A-A

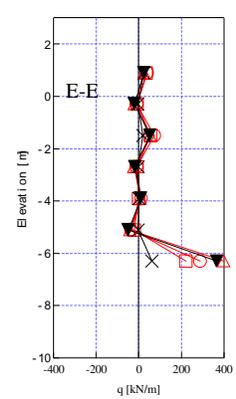|	Event	Time (sec)	Tension in tie-rod (kN/m)
×	before blasting	0.000	52.16
	max. tension in tie-rod	45.725	184.38
□	max. strain in sheet	45.760	150.21
△	max. strain in anchor pile	46.575	99.38
▼	after blast	90.000	90.21

B-B

	Event	Time (sec)	Tension in tie-rod (kN/m)
×	before blasting	0.000	10.90
	max. tension in tie-rod	45.730	123.14
□	max. negative strain in sheet pile	45.765	93.97
○	max. positive strain in sheet pile	45.770	89.81
△	max. strain in anchor pile	45.780	84.81
▼	after blast	90.000	50.64

E-E

	Event	Time (sec)	Tension in tie-rod (kN/m)
×	before blasting	0.000	22.09
□	max. negative strain in sheet pile	41.315	76.70
	max. tension in tie-rod	42.040	85.45
○	max. positive strain in sheet pile	42.070	82.53
△	max. strain in anchor pile	42.930	61.24
▼	after blast	90.000	51.03

F-F

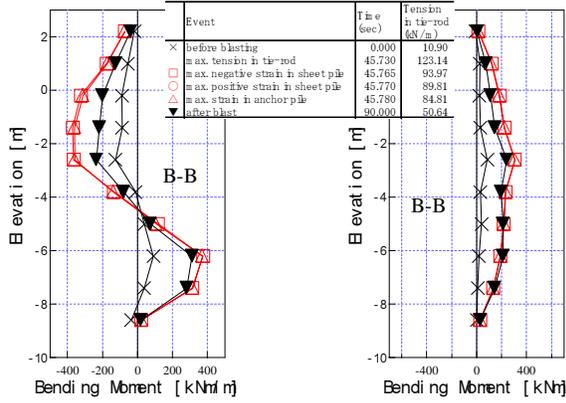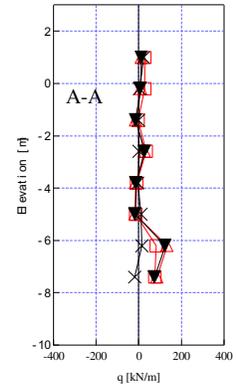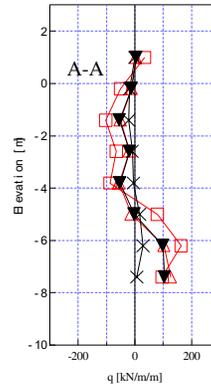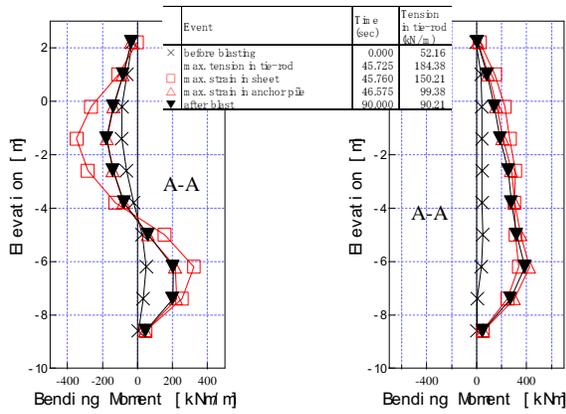|	Event	Time (sec)	Tension in tie-rod (kN/m)
×	before blasting	0.000	22.05
□	max. negative strain in sheet pile	40.590	115.81
○	max. positive strain in sheet pile	40.610	125.81
	max. tension in tie-rod	42.270	54.73
△	max. strain in anchor pile		
▼	after blast	90.000	50.31

Figure 15 Distribution of bending moment

Figure 16 Distribution load acting on sheet pile and anchor pile

landward direction. On the other hand, the deflection in shallow portions of sheet piles such as SPB-3, SPB-6 and SPE-3 was a little convex in landward direction at the beginning and became seaward convex largely in the sequence of blasting. The deflection at SPE-6 was also landward convex at the beginning but it did not became seaward convex during and after blasting.

Data from instruments LB and LE in Fig. 12 show the time histories of the tie-rod tensions in sections B-B and E-E, respectively. The tensions are plotted as absolute value. Tie-rod tensions began to fluctuate gradually after the beginning of blasting vibrations and varied sharply when the blasting progressed to the area behind the anchor piles. Since the blasting sequence was performed in the direction from back of the anchor piles to the sheet piles, anchor piles were pushed out seaward and tensions decreased gradually around 25 – 35sec. Especially in the case of blasting immediately in back of the anchor piles, tie-rod tensions decreased to 0.00. The load cells attached to the tie-rods can measure only tension. When the blasting moved to the area between steel sheet piles and anchor piles, tension began to increase with fluctuation. The residual tension was much larger than that prior to blasting.

Deformation of steel sheet piles and anchor piles
The bending moment distributions of sheet piles and anchor piles in each section are shown in Fig. 15. The absolute value is plotted in this figure; bending moment when the load did not act on sheet and anchor piles before pile placement is set to 0.00. Bending moment is calculated by multiplying the bending strain by section modulus and Young's modulus. The distributions at the following five events are shown in the figure.

1) Before blasting induced shaking
2) When the bending strain of the sheet pile was the maximum negative value
3) When the bending strain of the sheet pile was the maximum positive value
4) When the bending strain of the anchor pile was the maximum positive value
5) After blasting induced shaking

Bending moments of the sheet piles above the sea bottom were negative in all the sections, having a peak in the middle between the tie-rod attachment position and the sea bottom (see Fig. 15). On the other hand, bending moments under the sea bottom were positive. Therefore, this result indicates the same tendency as the model shake experiments by Arai[3]; the seabed in front of steel sheet piles resisted their displacement. When the blasting sequence reached the area behind the sheet piles, the bending moment of the sheet piles was maximized. The negative bending moments above the sea bottom fluctuated with blasting induced shaking and considerably diminished at the end of blasting. On the other hand, the positive bending moments under the sea bottom remained near the maximum value after the end of blasting shake and it is considered that influence of sheet pile displacement toward the sea was larger than that of fluctuation due to blasting. Absolute values of positive bending moments under the sea bottom were larger than that of negative moments above the sea bottom except for section A-A. The large passive resistance of the stiff seabed certainly appears to have acted on the embedded portion of sheet piles. Since the sheet piles of the non-seismic designed quay were embedded more shallowly than those of the seismic designed quay, the position of reversal from positive to negative in the moment distribution and that of the negative maximum moment occurred at a higher elevation than those of the seismic designed quay. Additionally, it is considered that the actual water depth may have been shallower than -4.8m as the reversal position from positive to negative existed at a depth of around -4m.

As shown in Fig. 15, indicating tension in the tie-rods per unit depth, the tensile force loaded the maximum value almost as soon as the bending stain of the sheet piles was maximized. The maximum tensions were different dependent on the sections and many factors appeared to influence tension in the tie-rods.

This may be due to the fixed condition of anchor piles, the initial tension at installation, the relative position between blasting holes and measurement devices, and so forth.

The bending moment of the anchor pile was maximized 0.8 – 1.7sec later than the time when that of sheet pile was maximized (at almost the same time only in section B-B, Fig. 15). It is suspected from the moment distribution that tie-rod tension acted on anchor piles as cantilever fixed at the boundary between the stiff original seabed and reclaimed soil; horizontal resistance did not act on anchor piles at the time due to liquefaction of the reclaimed soil.

Distribution load obtained from bending moment data is shown in Fig. 16, which gives the second order finite differential of bending moment with strain gauge interval along pile length. Loads toward the sea and land are made negative and positive respectively in this figure. Earth pressure from the backfill and subgrade reaction from the seabed acted on the sheet piles above and below the sea bottom, respectively. For both the seismic and non-seismic designed quays, a larger amount of the distributed load during the test acted than before the blasting induced shaking. Large load remained after the blasting in the embedded portion of sheet piles. In contrast, the residual earth pressure above the sea bottom was not as large as the earth pressure before the experiment. Large load acted and remained in the anchor piles after the blasting in the portion under the depth of -6m.

As evidenced by the time history of pore water pressure in Fig. 12, liquefaction had occurred in the shallow portion of backfill; therefore, the resistance by earth pressure to anchor pile deformation was not as strong as shown in Fig. 16. On the other hand, liquefaction did not occur below the depth of -6m and it is found that only this portion of ground was sustained the increased tension in tie-rods. For the anchor piles below the depth of -6m, larger load acted in the non-seismic designed quay because the anchor piles were shorter and fewer than those in the seismic designed quay; see Figs. 1 and 6.

The function of anchor piles, which is to sustain the deformation of sheet piles, diminished in the process of liquefaction of the backfill as mentioned above and only the sheet piles sustained earth pressure acting on them. Specifications of the sheet pile, such as sectional shape and length, were almost equal in the seismic and non-seismic designed quay walls. Accordingly it is considered that small difference between the quay walls, as shown in Fig.13, arose from these factors.

CONCLUDING REMARKS

The dynamic behavior of a full-scale steel sheet pile quay wall during excitation was investigated in this experiment with use of controlled blasting to induce liquefaction. Conclusions derived from this study are summarized as follows:

1) By producing vibration with controlled blasting, the behavior of full-scale steel sheet pile quays was observed during liquefaction. The maximum acceleration during blasting exceeded 10G and the waveform of acceleration by the controlled blasting was quite different from that of ordinary earthquakes. However, excess pore water pressure ratio reached about 1.0 and it is thought that the backfill almost completely liquefied.

2) The displacement of a steel sheet pile quay stopped with the end of blasting. Since the anchor piles were connected to tie-rods their displacement was nearly equal to that of sheet piles; they responded in conjunction with the sheet piles.

3) The steel sheet piles were seaward convex over the sea bottom and landward convex under the sea bottom. The absolute value of bending strain under the sea bottom was mostly larger than that over

the sea bottom. Subgrade reaction to anchor piles above the depth of around 6m decreased due to liquefaction and below this level increased to sustain the tie-rod tension.

4) The function of anchor piles diminished in the process of liquefaction of the backfill and only the sheet piles sustained earth pressure acting on them. Specifications of the sheet piles were almost equal in the seismic and non-seismic designed quay walls. Accordingly it is considered that small difference between the displacements of the quay walls arose from these factors.

ACKNOWLEDGMENT

REFERENCES

1   Sugano, T., Kohama, E., Mitoh, M. and Shiozaki, Y.: Seismic performance of urban, reclaimed and port areas - full scale experiment at tokachi port by controlled blasting technique, Proc. of the 11th Japan Earthquake Engineering Symposium, pp.901-906, 2002 (in Japanese).

2   Sasaki, K., Ueda, M., Yamazaki, M. and Sugano, T.: Proof experiment using artificial earthquake concerning the effect of permeable grouting method against liquefaction, the proceeding of the 57th JSCE Annual Meeting, 2002 (in Japanese).

3   Arai, H.: Vibration test of model anchored sheet-pile walls – a case of movable anchor walls –, Technical Note of the Port and Harbour Research Institute, Ministry of Transport, No.296, 1978 (in Japanese).

4   Japan Road Association: Specifications for Highway Bridges, Part IV; Substructures, 2001.