SHAKING TABLE TESTS ON THE MECHANISM OF LIQUEFACTION-INDUCED GROUND FLOW BEHIND QUAY WALLS

Key HIGUCHI[1], KeySusumu YASUDA[2], Tomohiro TANAKA[3], Daisuke ARAI[4] And Toshio KOBAYASHI[5]

SUMMARY

Shaking table tests were carried out to demonstrate the mechanism of the liquefaction-induced ground flow behind sea walls. Two types of sea walls, a caisson type quay wall and a sheet pile type river revetment, which were damaged during the Hyogoken-nambu and Niigata earthquakes, were selected for the tests. The caisson type wall moved quickly and the sheet pile type wall tilted gradually after the occurrence of liquefaction in the tests.

INTRODUCTION

Many quay walls moved towards sea and very large ground displacement occurred behind the walls due to liquefaction in and around Kobe City during the 1995 Hyogoken-nambu earthquake in Japan. For example, the average horizontal and vertical displacements of quay walls in Port Island were 2.7 m and 1.3 m, respectively. The ground flow behind the walls brought severe damage to many structures, such as bridges, buildings and lifelines. The area to which flow expanded was almost 100 to 200 m behind the walls. About 30 years ago, revetments of Shinano River were also moved towards the river and the ground behind the revetments flowed towards the river in Niigata City during the 1964 Niigata earthquake. The maximum displacement of the ground near the river revetments was more than 8 m. And the maximum area to which flow expanded was more than 200 m. Two types of sea walls, a caisson type quay wall and a sheet pile type river revetment, were selected for the tests. Several shaking table tests were carried out to demonstrate the mechanism of the ground flow behind the walls.

TEST EQUIPMENTS AND METHOD

Tests for the caisson type quay wall

Figure 1 shows the model for the caisson type quay wall which simulates a quay wall at Uozaki-hama in Kobe. The soil container used was 220 cm in length, 50 cm in depth and 45 cm in width as shown in Fig.1. Model for the caisson was 57mm in width, 110mm in height, 247mm in length and 20.1kN/m^3 in weight. Four piles of 12mm×10mm in width and 260mm in length made by acrylic fiber were fixed on the bottom of the container, as a model for the pile foundation of the highway bridge at Uozaki-hama. A model footing of 210mm×160mm in width and 40mm in thickness was fixed on the top of the piles. Clean sand named Toyoura sand was used to construct the model ground. The relative density of liquefiable layer and lower non-liquefiable layer were adjusted as 40 % and 70%, respectively. A vinyl sheet was sandwiched between the liquefiable layer

[1] *Dept. of Civil Eng., Tokyo Denki University, Saitama, Japan Fax No:+81-492-96-6501*
[2] *Dept. of Civil Eng., Tokyo Denki University, Saitama, Japan Fax No:+81-492-96-6501*
[3] *Shin-gijyutsu-koei Co. Ltd., Fukuoka, Japan*
[4] *Dept. of Civil Eng., Tokyo Denki University, Saitama, Japan Fax No:+81-492-96-6501*
[5] *Dept. of Civil Eng., Tokyo Denki University, Saitama, Japan Fax No:+81-492-96-6501*

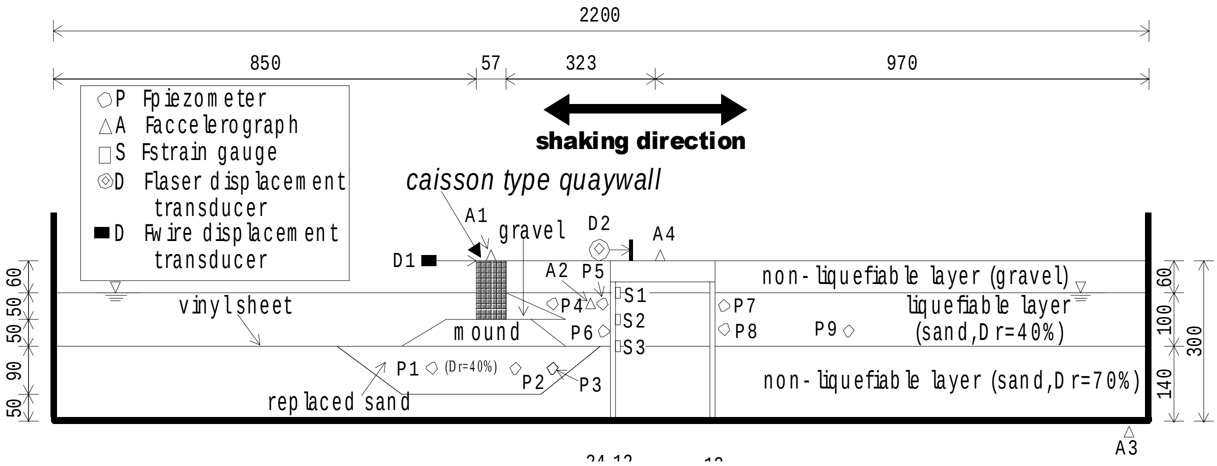


Fig.1 Soil container and model of the caisson type wall

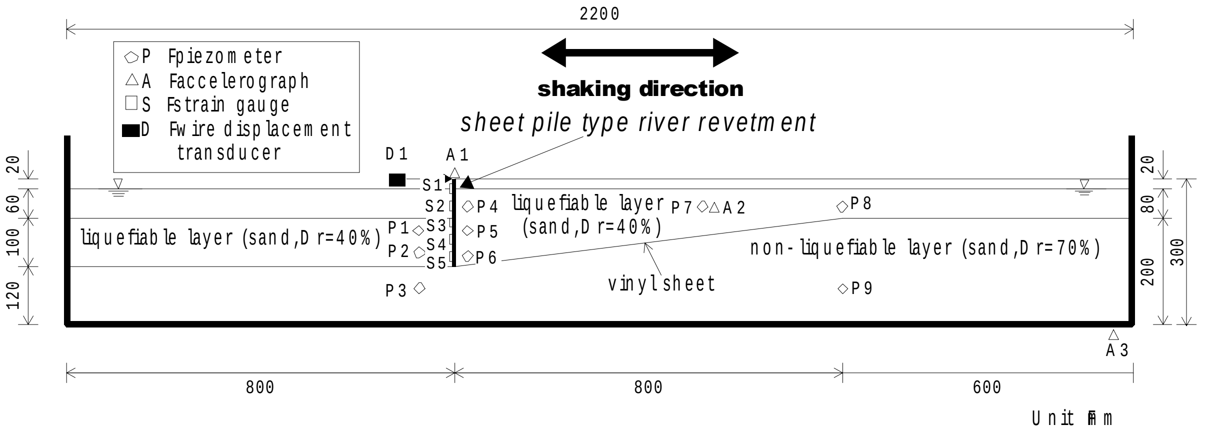


Fig.2 Soil container and model of the sheet pile type wall

Table 1 Test case

Type of wall	Test No.	Input acceleration
Caisson	1-1	270gal
	1-2	210gal
	1-3	190gal
	1-4	160gal
	1-5	130gal
Sheet pile	2-1	150gal
	2-2	110gal
	2-3	80gal

were 100mm and 60mm. Reduction scales in length and weight were about 1/100 and 1/1, respectively.

Nine piezometers and one accelerograph were installed in the ground. Two pair of accelerographs and displacement transducers were placed on the caisson and the footing. Three pairs of strain gauges were put on the surface of the piles. Shaking motion with sine wave at 3Hz was applied for 10 seconds. Five tests with different amplitude of shaking were carried out as shown in Table 1.

2.2 Tests for the sheet pile type river revetment

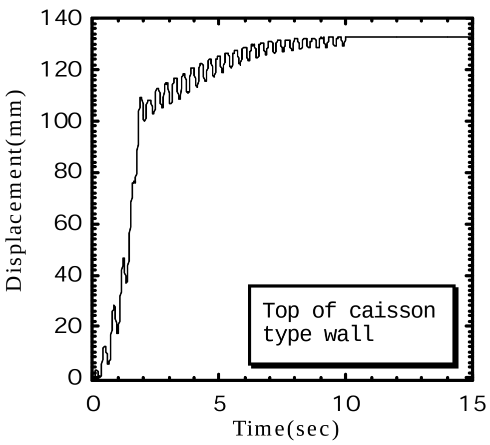
Figure 2 shows the model for the sheet pile type wall which simulates the river revetment at Showa Bridge in Niigata City. Soil container and sand used were same as those used in the test for the caisson type wall. A steel plate with 1.2mm thick, 180mm high and 447mm wide, was used for the model of the sheet pile type wall. Relative densities of a liquefiable layer and lower a non-liquefiable layer were 40% and 70%, respectively. A Vinyl sheet was sandwiched between these layers as same as the test for the caisson type wall. A pare of Teflon sheet were pasted between the edge of the steel plate and side wall of the container, to prevent the outflow of liquefied sand.

Nine piezometers and one accelerograph were installed in the ground. One accelerograph and one displacement transducer were placed on the top of the steel plate. Five pairs of strain gauges were put on the steel plate. Pins were put on the ground surfaced with the interval of 5cm to measure the distribution of displacements on the ground surface. Shaking motion with sine wave at 3Hz was applied for 10 seconds. Three tests with different amplitude of shaking were carried out as shown in Table 2.

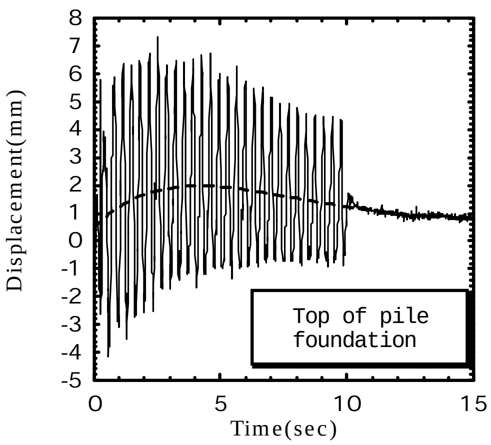
3. TYPICAL TEST RESULTS

As a typical test result for the caisson type quay wall, time histories of excess pore water pressures, displacements of the caisson type wall and displacement of the top of pile foundation of Test No.1-2, are shown in Fig.3. Positive values in the displacements show the displacements toward the sea. Based on Fig.3(c), it can be judged that liquefaction occurred at about 0.5 seconds because the excess pore pressures ratio in the liquefiable layer increased up to about 1.0. On the contrary, the excess pore water pressure ratios in the gravel and the replaced sand increased up to 0.8 to 0.9 and 0.1 to 0.2 only at the end of shaking. The reason why the pore water pressure ratio in the replaced sand was less than 1.0 is discussed later. The wall moved quickly in 1 to 2 seconds as show in Fig.3(a). The amplitude of the displacement of the top of pile foundation increased at the time of the occurrence of liquefaction. Then the amplitude decreased. Broken line in Fig.3(b) shows static displacement of the pile foundation. The static displacement increased with the movement of the wall and reached the maximum value at about four seconds. Then the static displacement decreased. Some residual displacement remained after the shaking.

As a typical test result for sheet pile type wall, time histories of excess pore pressure ratios and displacement of the top of the wall are shown in Fig.4. The excess pore water pressure ratios reached about 1.0 at around 0.5 seconds and did not decrease up to the end of shaking. Therefore, it is estimated liquefaction occurred at about 0.5 seconds and was sustained up to the end of shaking. The sheet pile type wall tilted gradually after the

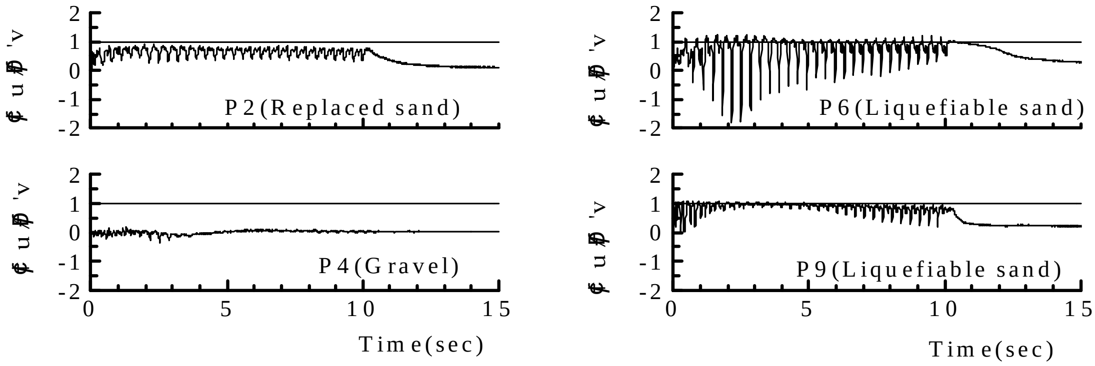


(a) Displacement of wall



(b) Displacement of pile foundation

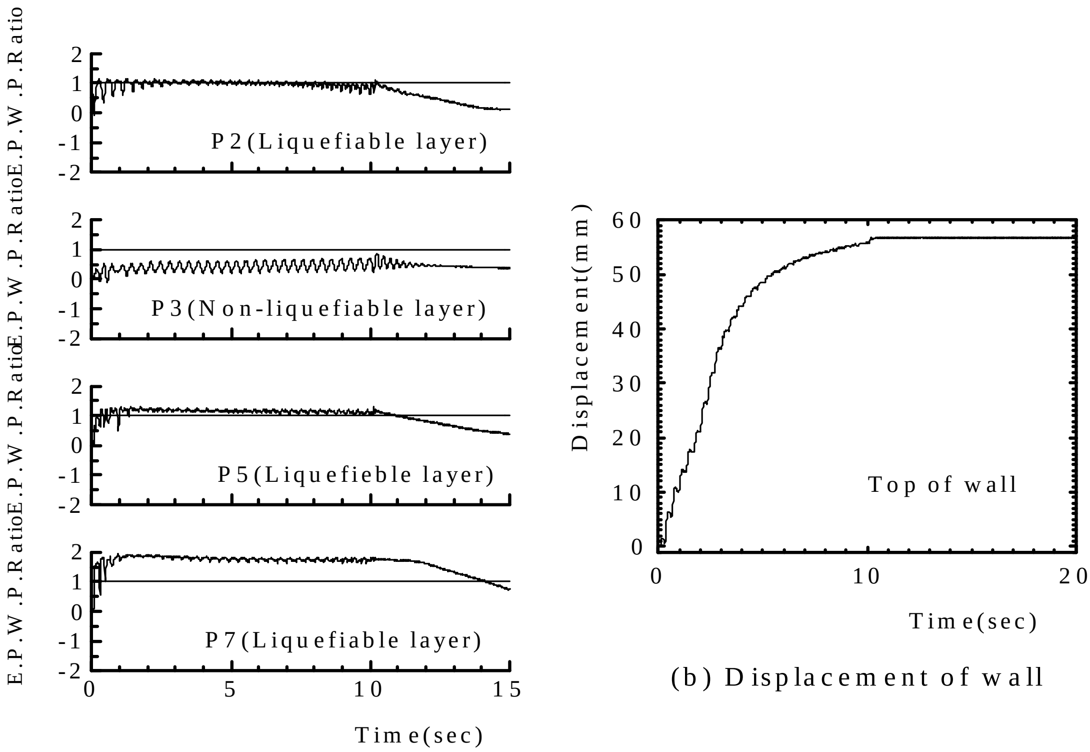
Fig.3(a),(b) Typical test result for the caisson type wall



(c) Excess pore water pressure

Fig.3 (c) Typical test result for the caisson type wall

occurrence of liquefaction as shown in Fig.4(b). It is noted that the speed of movement of the caisson type wall and the sheet pile type wall were quite different as shown in Fig.3(b) and Fig.4(b).



(b) Displacement of wall

(a) Excess pore pressure ratio

Fig.4 Typical test result for the sheet pile type wall

4. LIQUEFIED PARTS IN THE GROUND

Table 2 compares liquefied parts in the ground for each test. Open circles and cross marks show liquefied parts and non-liquefied parts judged form excess pore pressure ratio. In the caisson type wall, liquefaction occurred in the liquefiable ground if the input acceleration was greater than 190gals, because the pore water pressure ratios at P6 to P9 increased up to 1.0. On the contrary, the ratios at P2 and P3 in the replaced sand did not increase up to 1.0 though the input acceleration was high. It seems that the ratios were less than 1.0 because high initial shear stress and overburden pressure acted in the replaced sand due to the weight of the caisson. The reason why the

Table 2 Liquefied parts in the ground

Type of wall	Test No..	Input acceleration	P1	P2	P3	P4	P5	P6	P7	P8	P9
Caisson	1-1	270gal	○	×	×	×	×	○	○	○	○
	1-2	210gal	○	×	×	×	×	○	○	○	○
	1-3	190gal	○	×	×	×	○	○	○	○	○
	1-4	160gal	×	×	×	×	×	×	×	×	×
	1-5	130gal	×	×	×	×	×	×	×	×	×
Sheet pile	2-1	150gal	○	○	×	○	○	○	○	○	×
	2-2	110gal	○	○	×	○	○	○	○	×	×
	2-3	80gal	×	○	×	○	○	×	○	○	×

○: Excess pore pressure ratio increased up 1.0,
×: Excess pore pressure ratio did not increase up to 1.0

ratios at P4 and P5, which were close to the gravel, did not reached 1.0 must be the dissipation effect through the gravel.

In the sheet pile type wall, liquefaction occurred in whole liquefiable layer if the input acceleration was greater than 110gals. Some part did not liquefied under the low input acceleration of 80gals.

5. RELATIONSHIPS BETWEEN MOVEMENT OF WALLS AND INPUT MOTION

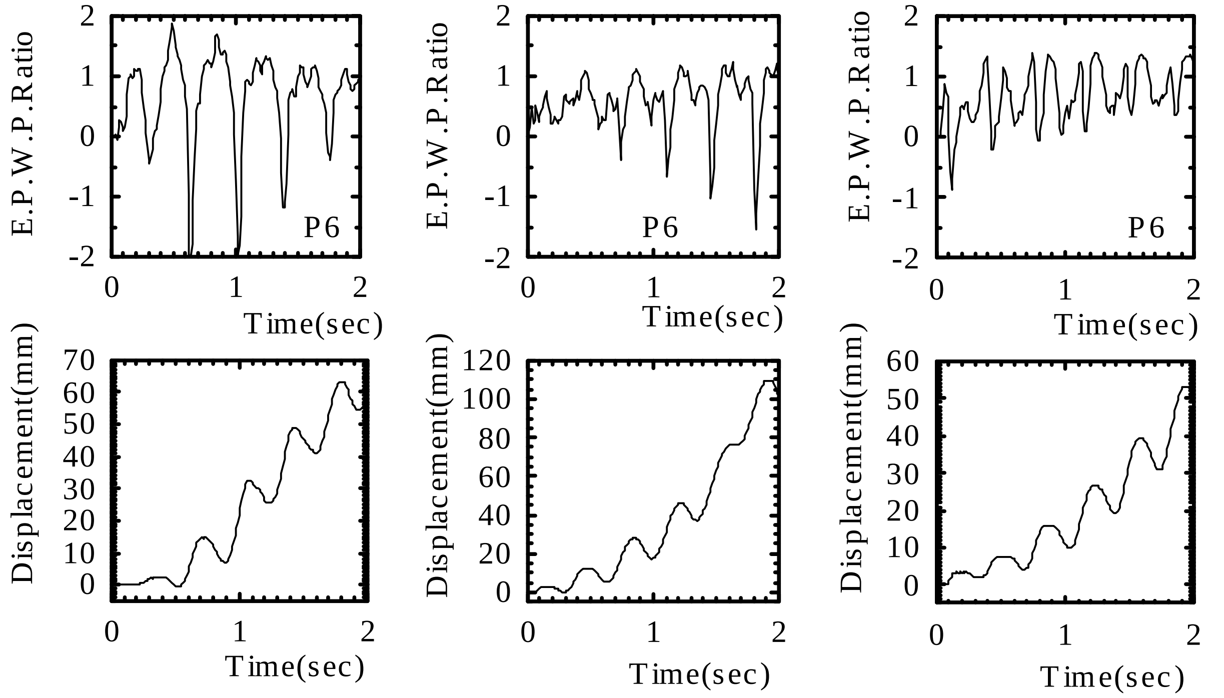
5.1 Caisson type wall

Figure 5 shows the detailed time histories of the displacement of the top of the wall(D1) and pore water pressure ratio in the ground behind the wall(P6) during the beginning 2 seconds, under the input acceleration of 190, 210 and 270gals. As shown in the figure, negative pore pressure occurred in the ground behind the wall. Figure 6 compares pore water pressure, displacement of the top of the wall, displacement of the top of the footing and differential displacement between the top the wall and the top of the foundation. If the displacement of the footing is assumed as similar as the displacement of the ground surface, it can be said that the negative pore water pressure occurred at the time when the differential displacement between the wall and the ground increased. The caisson type wall can move solely due to inertia force. Therefore negative pore pressure must be induced when the wall becomes separate from the ground. The value of the negative pore water pressure increased with input acceleration, as shown in Fig.5, because the inertia force increased with the input acceleration.

Figure 7 compares time histories of the displacement of the top of the wall in different input accelerations. The speed of the movement of the wall increased with the input acceleration. Residual displacement also increased with the input acceleration. Figure 8 shows relationship among the input acceleration, the residual displacement at 40 seconds and the maximum displacement. The maximum displacements were almost same as the residual displacement. The residual displacements were large if the input acceleration was greater than 190gals though the displacements were almost zero if the acceleration was less than 160gals. This means that the residual displacement induced by not only the inertia force but also the increased earth pressure due to liquefaction.

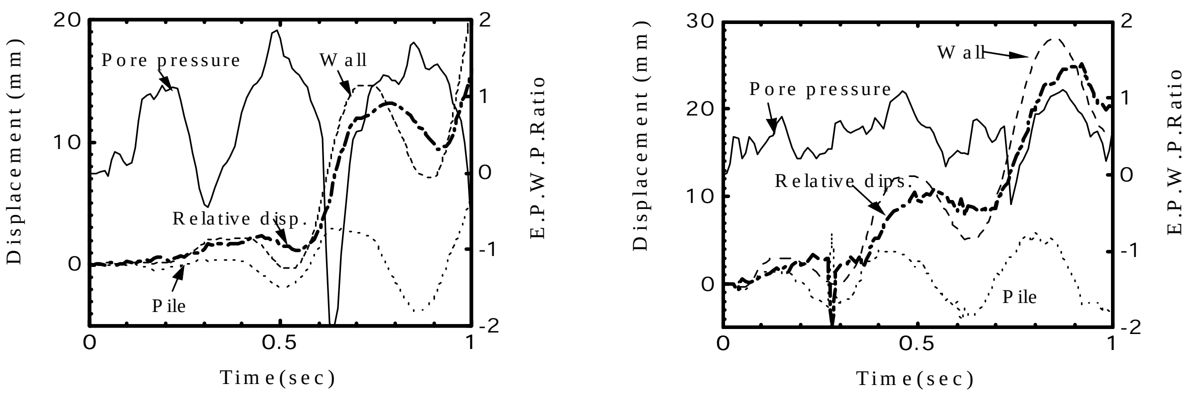
Figure 9 shows time histories of the displacement of the top of the foundation. Amplitude of displacement and static displacement increased with input acceleration. Figure 10 shows relationships among the input acceleration, the maximum displacement and residual displacement. As shown in the figure, the maximum displacement was greater than the residual displacement. This means that the ground pushed the pile foundation strongly during the flow of the ground toward the sea, and the piles were rebounded slightly after the stop of the flow. If the amplitude of the displacement is included, the maximum displacement was much greater than the residual displacement.

Figure 11 shows distributions of stains of the piles in depth at the time when the strain at S2 became the maximum. Positive value in the figure means the pile was convex towards the sea. The strain was very small if the ground did not liquefied. And the strain increased with input acceleration when liquefaction occurred.



(a) Test No.1-1(270gals) (b) Test No.1-2(210gals) (c) test No.1-3(190gals)

Fig.5 Detailed time histories of displacement of wall and pore water pressure



(a) Test No.1-1(270gals) (b) Test No.1-2(210gals)

Fig. 6 Comparison of displacement and pore water pressure

5.2 Sheet pile type wall

Though figures are not shown here, negative pore pressure did not occur when the sheet pile type wall tilted towards the sea. The difference of the occurrence of the negative pore pressure between the sheet pile type and caisson type wall must be attributed to the difference of the inertia force.

Figure 12 compares time histories of the top of the wall in different level of shaking. Speed of the movement of

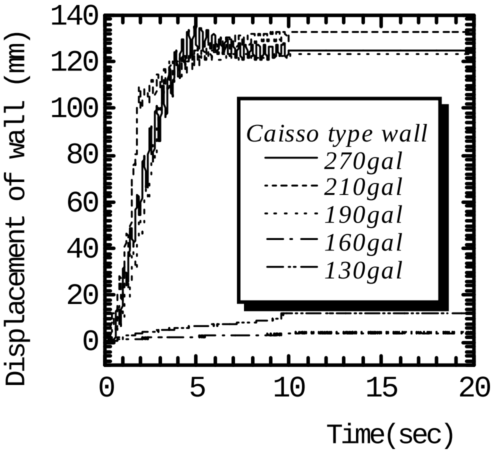


Fig.7 Time histories of displacement of wall

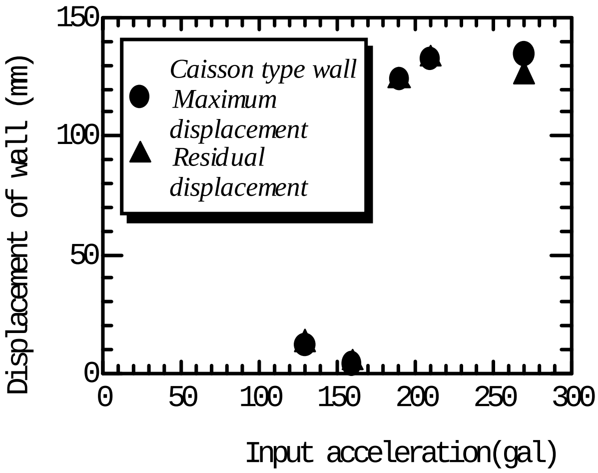


Fig.8 Relationship between displacement of wall and input acceleration

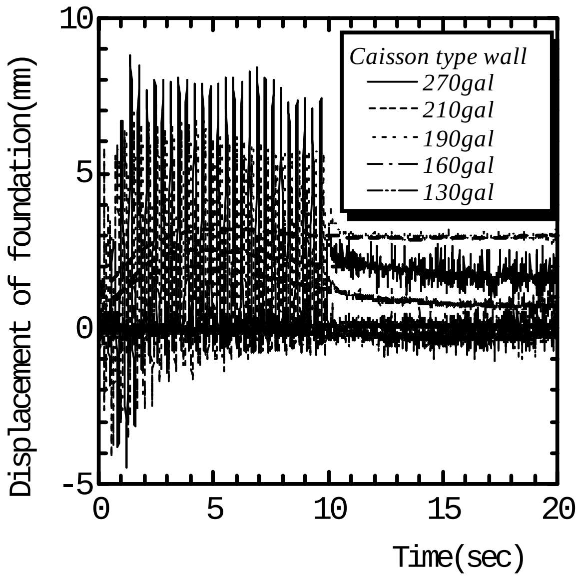


Fig.9 Time histories of displacement of foundation

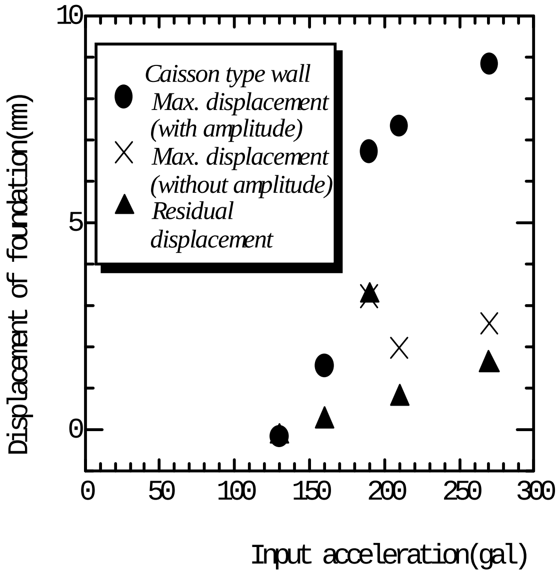


Fig.10 Acceleration vs. displacement of foundation

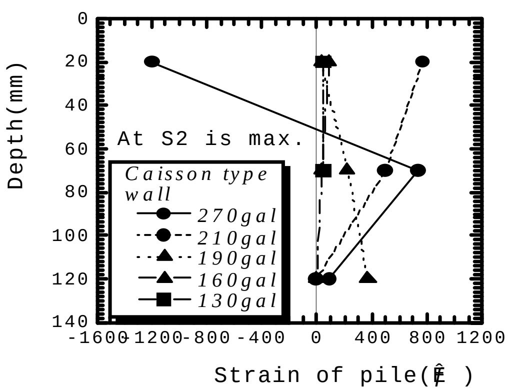


Fig.11 Distribution of strain of pile in depth

the wall and residual displacement increased with the input acceleration. As show in Fig.13, the residual displacements in 80gals and in 110gals were quite different. The wall tilted fairly in 110 gals though the wall tilted for only 4 degrees in 80 gals. The residual displacement in 80gals must be small because some parts in the ground did not liquefy as mentioned before.

Distributions of stains of the wall in depth at the time when the strain at S1 became the maximum are shown in Fig.14. Positive value in the figure means the wall was convex towards the sea as same as Fig.11. The strain was small if some parts did not liquefied under the low level of shaking.

Figure 15 shows distribution of horizontal displacements on the ground surface measures by pins after the stopping of shaking. The horizontal displacement decreased with the distance from the wall. And, the horizontal displacement at a distance increased with the input acceleration.

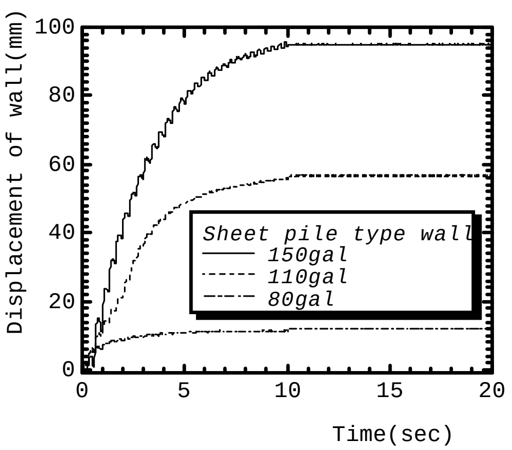


Fig.12 Time histories of displacement of wall

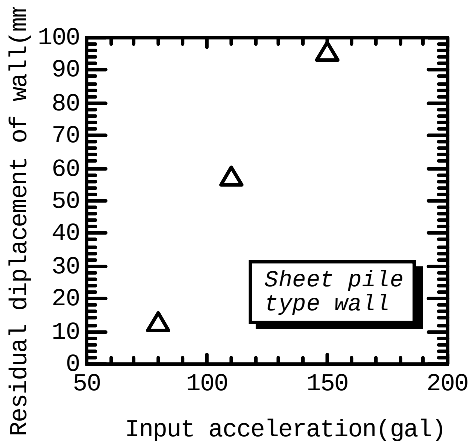


Fig.13 Relationship between input acceleration and residual displacement of wall

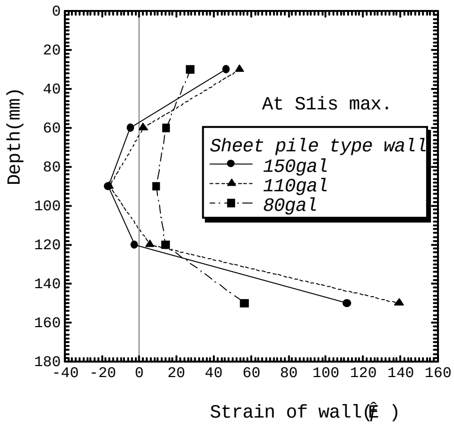


Fig.14 Distribution of strain of wall

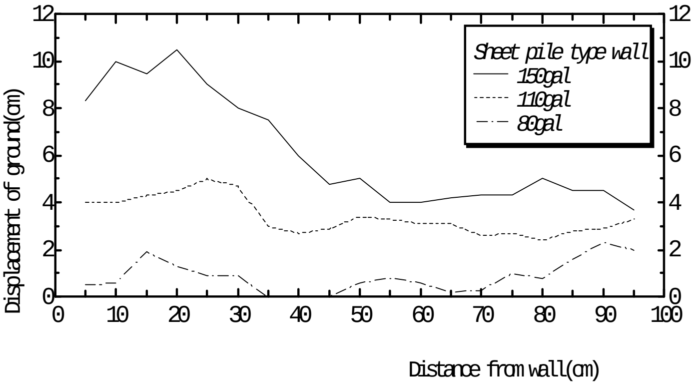


Fig.15 Distribution of displacement on the ground surface

6. CONCLISIONS

Several shaking table tests were carried out to demonstrate the mechanism of the liquefaction-induced ground flow behind two types of sea walls, and the following conclusions were derived:
(1) The caisson type wall moved quickly and the sheet pile type wall tilted gradually after the occurrence of liquefaction.
(2) Negative pore pressure was measured in the ground behind the caisson type wall.
(3) The maximum and residual displacements of the walls and foundation were quite large and increased with input acceleration if the ground liquefied.

ACKNOWREDGEMENTS

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RFERENCE

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