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A PROJECT FOR THE FIELD OBSERVATION OF SEISMIC BEHAVIOR OF FULL-SIZED TEST GRAVITY TYPE QUAY WALL

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SUMMARY

A field observation of dynamic behavior of full-scaled test gravity type of quay wall during earthquakes is under action in the Port of Kushiro, Hokkaido, Japan. This project aims to reveal the mechanism of the damage to quay walls associated with the earthquake induced liquefaction of the backfill ground, and provide information for the evaluation of earthquake earth pressure on retaining structure and the development of rational seismic design method for gravity type quay wall. Some records of the dynamic behavior of the test quay wall have been obtained so far, including that during the 2003 Tokachi-oki Earthquake, where the test quay wall was shook with the seismic intensity of rank III. In this paper the outline of the project is introduced, and the structure of the test quay wall and the observation system for the dynamic behavior are explained. And the vibration behavior of the test quay wall during the 2003 Tokachi-oki earthquake is briefly presented.

INTRODUCTION

Recently many structures have been damaged in destructive earthquakes. Especially Some types of infrastructures are often destroyed or damaged severely by earthquake induced liquefaction on riversides and port and harbor areas. In this case the lateral flow of liquefied ground plays an important role; that is, the lateral flows often thrust structural foundations and underground pipeline networks. In port and harbor areas, the lateral flow is triggered by the failure of quay wall. Then, the protection of quay walls is a pressing subject not only for the preservation of quay wall formation itself, but also for the protection of structures on backfill ground.

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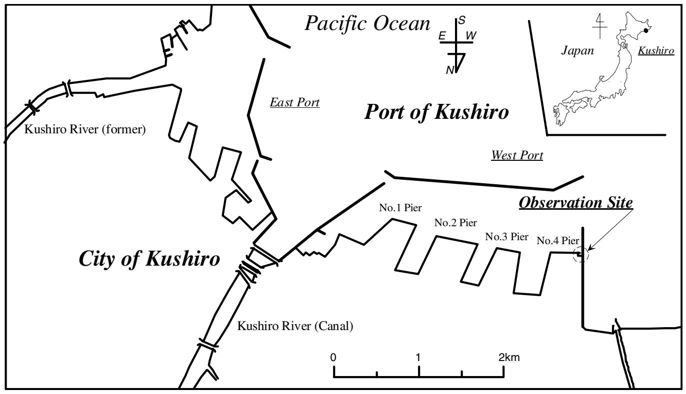

Fig. 1. The Port of Kushiro, Hokkaido, Japan.

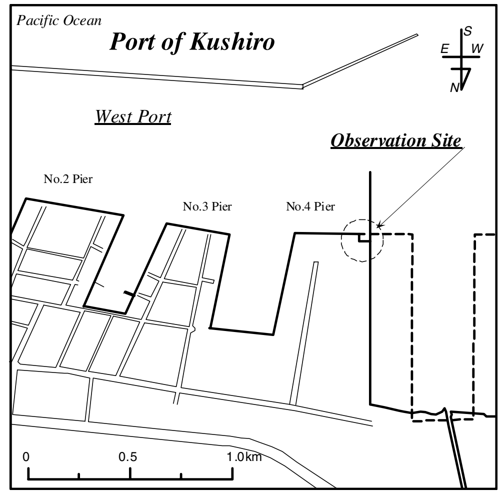

Fig. 2. Location of observation site; No. 4 Wharf in the west district of the Port of Kushiro.

The failure of gravity type of quay wall, that is, its excessive subsidence and/or displacement toward the sea, must be caused mainly by the corporation of inertial force and earth pressure from backfill ground. And the liquefaction of backfill ground is a decisive influence factor for the damage; it seems that this kind of failure was enhanced by the occurrence of liquefaction in the backfill in past many destructive earthquakes (e.g., JGS, 1994; Inagaki, et al. [1]; Kamon, et al. [2]). Actually, close examination of the damage to quay walls in Hokkaido, Japan, during three big earthquakes, i.e., 1993 Kushiro-oki, 1993 Hokkaido Nansei-oki and 1994 Hokkaido Toho-oki earthquakes, clarified that the degree of damage is strongly related to the occurrence of liquefaction in the backfill ground (JGS [3]; Hokkaido Development Bureau [4]). A clear proof of the effect of liquefaction on the stability of quay walls is seen in the damage to treated quay walls. Remedial measures against liquefaction have been positively made in Hokkaido Island after the 1983 Nihonkai-chubu earthquake; the backfill ground of quay walls was improved by a combination of sand compaction pile and gravel drain methods. The treated quay walls could survive large earthquakes with little or no damage, in marked contrast to the severe damage frequently seen to untreated quay walls.

Kohama et al. [5, 6] and Miura et al. [7] conducted some series of model shaking table tests on model quay walls focusing on the effect of liquefaction in the backfill ground. The movement of the caisson was found to be quite different depending on whether liquefaction occurred in the backfill ground or not. The fluctuating earth pressure on the caisson suppressed the movement of the caisson when liquefaction did not occur. On the other hand, sliding of the caisson was enhanced since the fluctuating component of earth pressure and the inertial force coincide in phase angle, when liquefaction occurred in the backfill ground. The fluctuating earth pressure acting on the back wall of the caisson in the process to liquefaction was carefully observed in the model shaking tests. It was found that the amplitude of the earth pressure first decreased to a very small value because of the reduction of the stiffness of the backfill due to the excess pore water pressure generation, and then increased because the phase angle of the earth pressure changed by 180 degrees. This indicates that stability criteria of the caisson should be developed not by the onset of the liquefaction but by the sudden phase change.

The above mentioned findings about the interaction between the caisson and backfill ground and resulting behavior of the dynamic earth pressure on the caisson in the process of liquefaction provides a clear vision for the necessity of the remedial measures against liquefaction in seismic design and also for the rational evaluation method of the dynamic earth pressure. If the characteristic dynamic earth pressure in the process of liquefaction is clarified in the quay wall of actual size, the rational earthquake resistant design must be progressed; see Miura et al. [8].

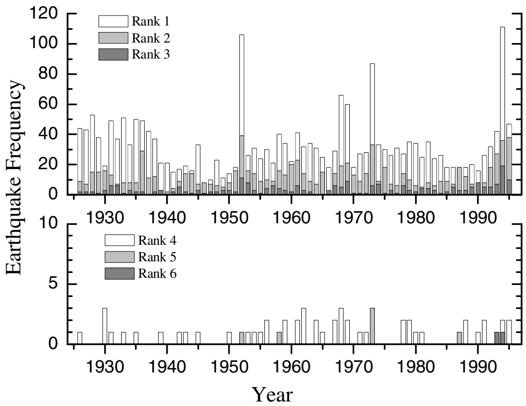

Fig. 3 Seismic activity of observation site

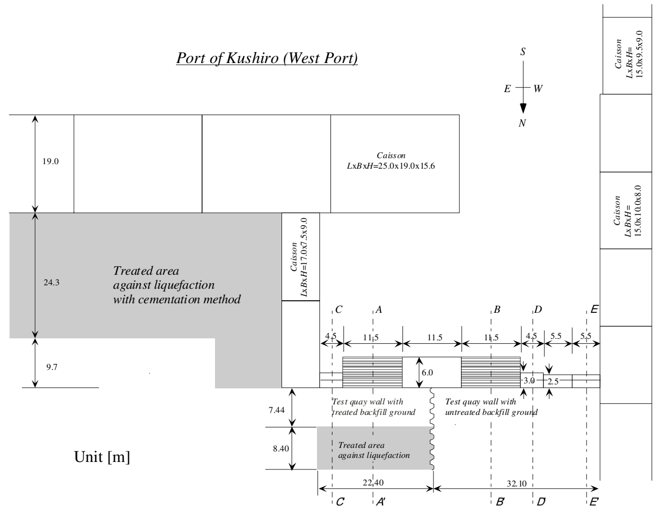

Fig. 4. Plane view of the test quay wall on No. 4 Wharf.

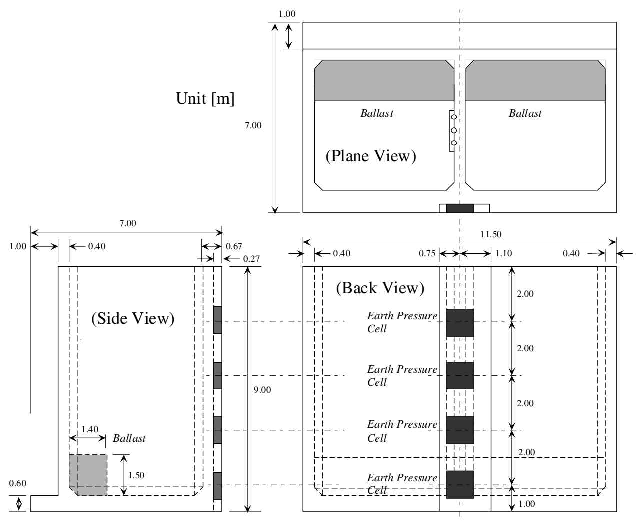

Fig. 5. Test caisson with four earth pressure cells on the back.

OBSERVATION SITE OF TEST QUAY WALL

The observation site for the dynamic behavior during earthquake of the full-sized test gravity type quay wall is located in the Port of Kushiro, Eastern Hokkaido, Japan; see Fig. 1. The Wharf No. 4 in the west district of the port is under development, and the tentative quay wall was constructed at the south side of the wharf as shown in Fig. 2; in the figure upward direction is designated as the south but not north. The test quay was constructed as a part of the tentative quay wall. The Port of Kushiro was selected for this projected, because the port has been steadily armored against earthquake induced liquefaction actively after 1986 Nihonkai-chubu Earthquake, and the awareness on earthquake disasters was extremely high in the authority of the Port of Kushiro. Also the seismic activity of the area is one of the highest in Japan. According to the statistical data of the earthquakes which shook this area for more than seventy years, it was expected that the earthquakes with seismic intensity of rank V may occur once for ten years, rank IV three times for four years, and rank III three or four times per year, as shown in Fig. 3. Actually for about three years from April 2001, the earthquakes with seismic intensity of rank III shook the observation site thirteen times, and those with rank IV and rank V+ shook once, respectively, including 2003 Tokachi-oki Earthquake, which occurred on 26 September, 2003.

TEST QUAY WALL AND CAISSONS

Figure 4 shows the plane view of the observation site. The temporary quay wall was constructed at the junction of the south side of the Wharf No. 4 and the break water on west side. Three caissons are arranged side by side in a row at 8m in nominal sea depth: two test caissons were placed at both east and west sides for the observation of their behaviors. The dimension of the test caisson is shown in Fig. 5. The shape of the test caisson is rather slender and possesses insufficient seismic resistance, compared with

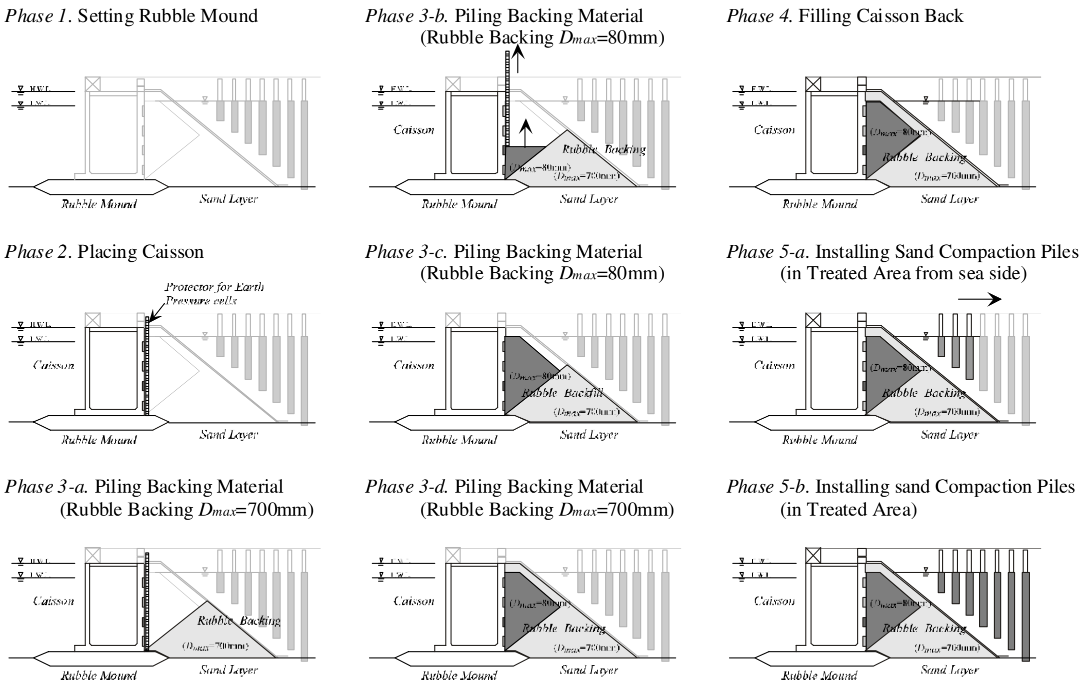

Fig. 6. Construction sequence of the test quay wall.

ordinary ones designed according to the contemporary seismic design code. They were designed intentionally so that the quake of the order of 100Gal causes notable slide of the caisson toward the sea in the frame work of seismic coefficient method regarding inertial force and earth pressure. The toe of the test caisson at seaside was employed to prevent the safety factor for rotation from falling short of that for slide. Four earth pressure cells with large sensitive panel of 1.0m x 1.0m were arranged vertically on the back of each of the test caissons, and then, some concrete ballast was placed at the seaside bottom of the caisson as a counter balance for the extra weight of the thick back wall and the earth pressure cells.

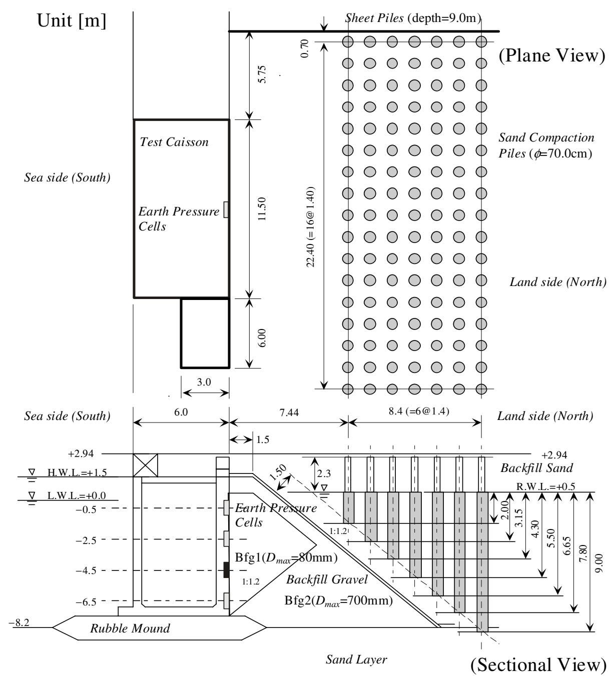

Fig. 7. Sectional view of backfill ground with remedial method installing sand compaction

The test caissons were cast on the ground, and then earth pressure cells, accelerometers and velocimeters were mounted in dry condition. The construction sequence of the quay wall is presented in Fig. 6. First, the seabed was excavated to a certain depth and leveled, and then a rubble mound was prepared with coarse gravel (Phase 1). The caissons with a steel plate cover over the earth pressure cells were towed by tugboats, and placed on the rubble mound (Phase 2). Next, Rubble backing was provided to release the residual water pressure induced by waves and tide; the dimension of the rubble backing was determined according to the ordinary specification in the area. First, coarse gravel of about 700mm in diameter (about 200kg in mass) was heaped near the test caissons (Phase 3-a), and next, fine gravel of less than 80mm in diameter were pored and heaped behind

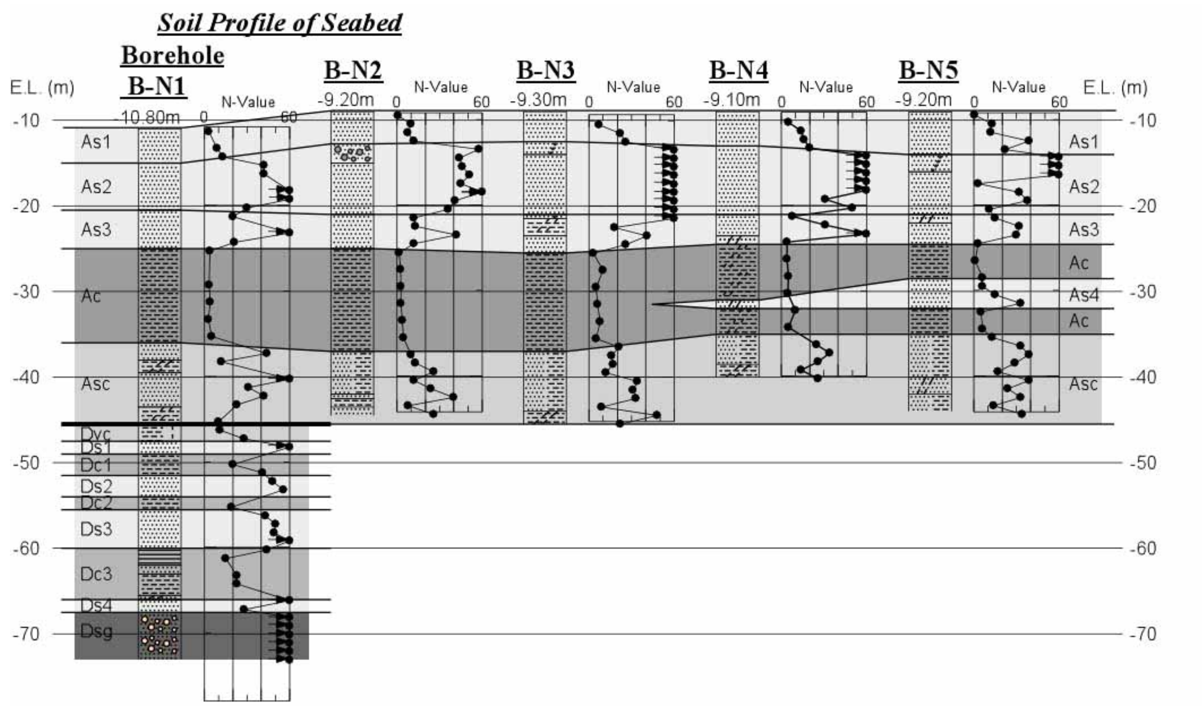

Fig. 8. Soil profile of seabed drawn from a series of standard penetration tests at observation site.

the test caissons removing the steel covers (Phase 3-b,c). The maximum diameter of the fine gravel was determined so as to maintain a certain accuracy of the earth pressure measurement by the earth pressure cell of 1.0m x 1.0m sensitive panel; The ratio of gravel diameter to panel dimension was determined to be less than 10, according to the previously conducted laboratory tests; see Miura et al. [9]. The gravel slope was covered with sand screen sheet (Phase 3-d), and the backfill ground was reclaimed with dredged fine sand (Phase 4).

To clarify the significance of liquefaction of backfill ground in the damage to quay wall during earthquake, the reclaimed backfill ground was first separated by a row of sheet piles of 9m in depth, and only the east part was treated against liquefaction with sand compaction pile method as shown in Fig. 4 (Phase 5-a,b). The sand compaction piles were designed based on the geological survey for soil profiling, standard penetration tests (SPT), the laboratory tests on undisturbed soil specimens for liquefaction strength, and the seismic response analysis with earthquake vibration of 350Gal in maximum acceleration. Specification of the provided sand compaction piles is that the pile diameter was 70cm, an interval was 1.4m in square arrangement, and the ratio of volume displacement was 0.19; see Fig. 7. An increase in N-value up to 20 was expected through the treatment.

GEOTECHNICAL CONDITION OF THE OBSERVATION SITE

Seabed

The soil profile at the observation site was drawn from a series of standard penetration tests (SPT); see Fig. 8. The shallow part of the seabed consists of an alluvial deposit of about 45 m in thickness, which is underlaid by a diluvial deposit. At EL -70m a consistent sandy gravel layer (Dsg) exceeding 60 in SPT N-

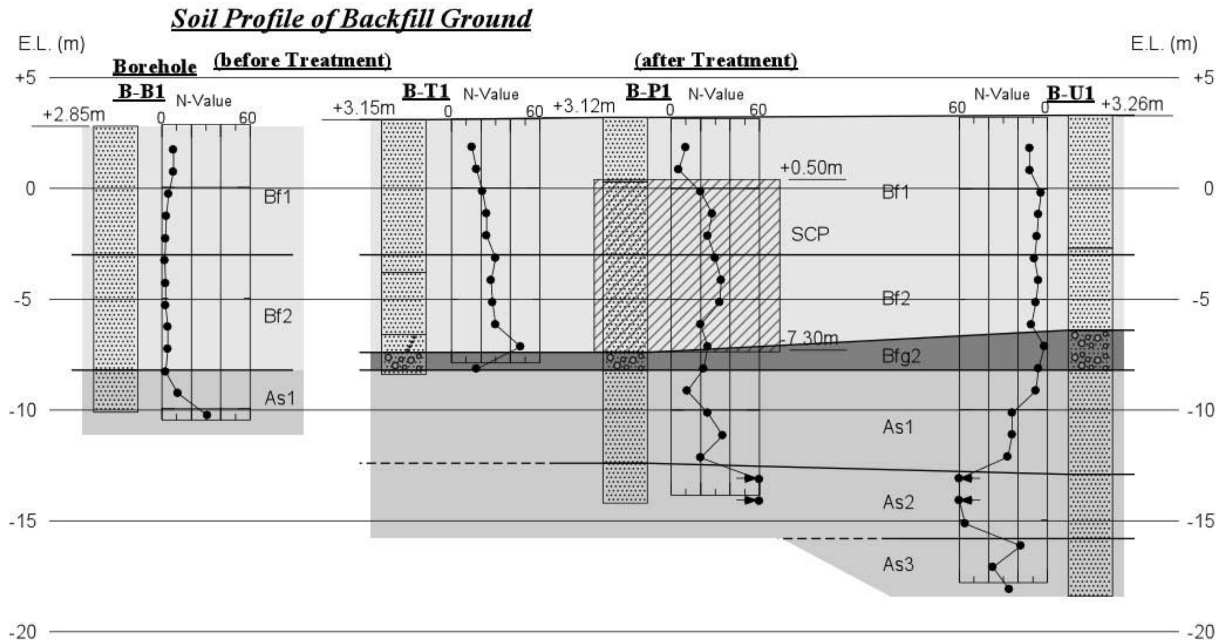

Fig. 9. Soil profile of reclaimed backfill ground before and after installation of sand compaction piles.

value was found, which can be dealt as a datum for engineering earthquake response analysis of the test quay wall.

The alluvial deposit is an uncemented Holocene deposit, and the upper part of the deposit consists of three sand layers (As1, As2, As3); layer As2 is relatively hard frequently exceeding 60 in SPT N-value. Layer As2 was considered as the base for bearing the weight of quay wall and reclaimed ground. These sand layers are underlaid by thick soft clay layer (Ac) of 14-15m in thickness and less than 5 in SPT N-value. Near the test caissons thin sand layer (As4) is inserted into layer Ac. The underlying diluvial deposit consists of several stiff layers including organic matter, clay, and gravel.

Owing to the multi-layer column model prepared for earthquake response analysis based on the geological survey, the natural vibration frequency was 1 Hz and 7 Hz in horizontal and vertical directions, respectively.

Reclaimed Backfill Ground

The soil profile of reclaimed backfill ground is presented with the results of SPT in Fig. 9; bore holes B-B-1, 2 were dug before the installation of sand compaction piles and bore holes B-P-1, 2, B-T1, 2 and B-P1, 2 were dug after the installation. To obtain information for vibration analysis, PS-wave velocities were measured. And the undrained cyclic triaxial tests were conducted on the intact soil samples obtained from the bore holes by means of triple-tube sampler or thin wall sampler, to evaluate liquefaction potential of the backfill ground. Through these field survey, laboratory tests and vibration analysis regarding liquefaction, the effect of remedial method with sand compaction pile method on the improvement of dynamic mechanical properties and liquefaction strength of reclaimed ground. Before the remedial method SPT N-value was only 4 over the reclaimed deposit, indicating the high possibility of liquefaction during earthquake. The N-value was, however, notably increased by the remedial method; N-value was more than 20 in the treated area.

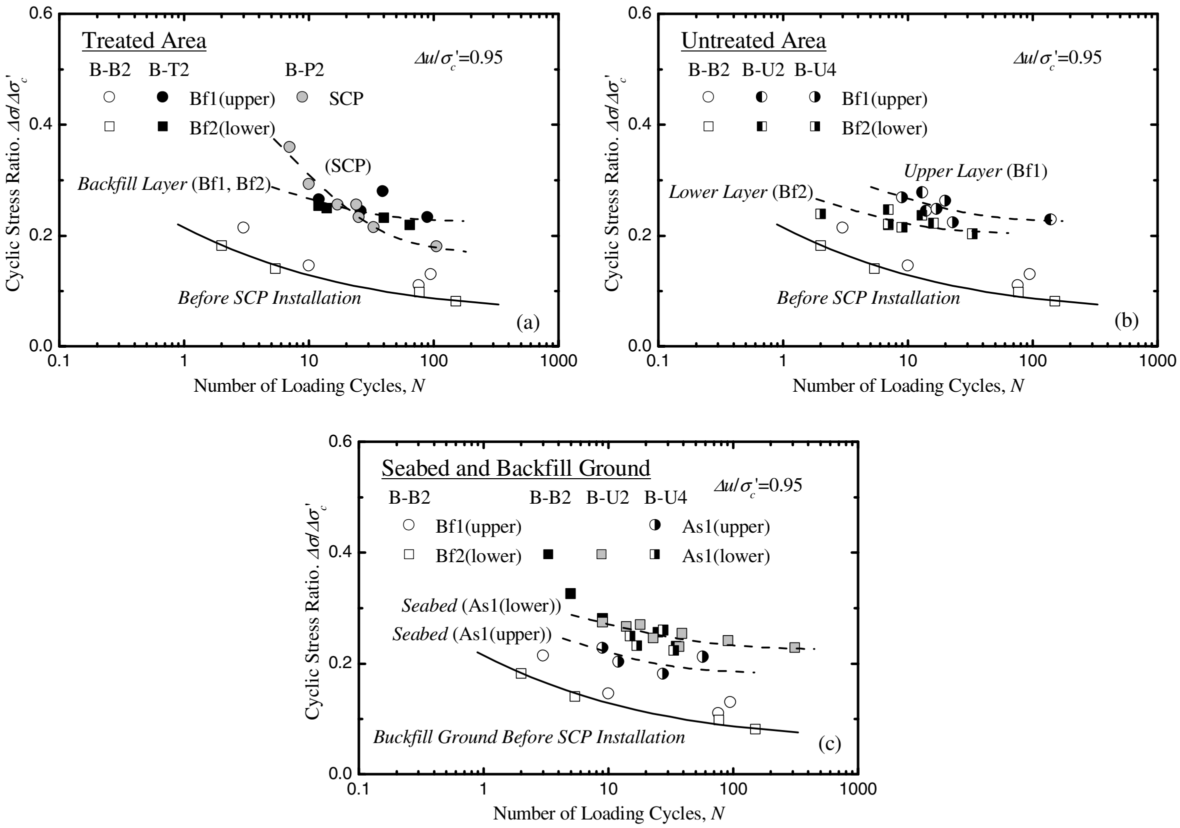

Fig.10. Liquefaction potential measured in undrained tri-axial tests on undisturbed samples; (a) Treated area, (b) Untreated Area, (c) Seabed surface layer As1

The change in liquefaction strength through the treatment against liquefaction, which was measured in some series of undrained cyclic triaxial tests, is presented in Fig. 10. As shown in Figs. 9(a,b), the liquefaction strength indicated by the shear stress ratio required to cause liquefaction in cyclic sinusoidal loading of 20 cycles was increased by about 0.1. The surface sand layer As1 of the seabed showed almost same values in liquefaction strength with the treated reclaimed ground.

SYSTEM FOR OBSERVATION

Arrangement of Sensors

The arrangements of sensors both in treated and untreated areas are shown in Fig. 10; character 'T' in the symbols is for treated area and 'U' is for untreated area.

Fourteen three-component accelerometers and six three-component velocimeters were arranged in the treated and untreated areas, to measure the vibration behavior of the test caissons and backfill ground. Each test caisson has the two accelerometers (TA1, 3 and UA1, 3) and three velocimeters (TV1, 2, 3 and UV1, 2, 3); velocimeters were employed for the measurement of not only velocity itself but also for the calculation of displacement. One accelerometer was placed in the rubble mound both in treated and untreated areas (TA2 and UA2). Vertical arrays of accelerometers were formed both in the treated and untreated areas (TA5, 6, 7, 8 and UA 5, 6). The deepest accelerometer (TA8) was placed in the base layer As2 at EL -14.0m.

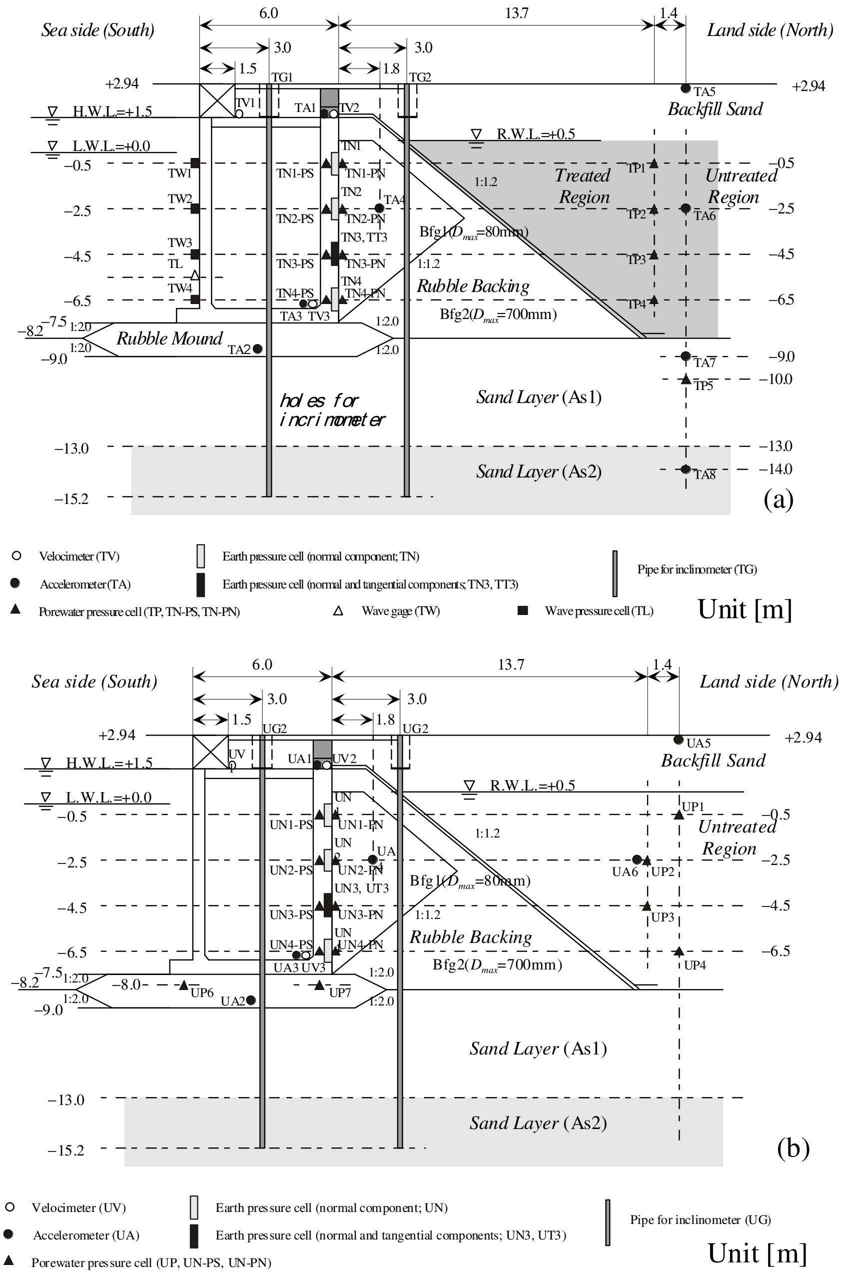

Fig. 11. Arrangement of sensors: (a) in treated area; (b) in untreated area

Pore water pressure cells were also arranged in the treated and untreated areas in vertical arrays, to measures the dynamic pore water pressure during earthquake (TP1, 2, 3, 4, 5 and UP1, 2, 3, 4). In the case of large earthquake, cumulative pore water pressure generation would be measured in the process to liquefaction. Two pore water pressure cells (UP6 and UP7) were placed also in the rubble mound to monitor the conductivity of wave pressure through the mound and the dynamic uplift pressure during earthquake.

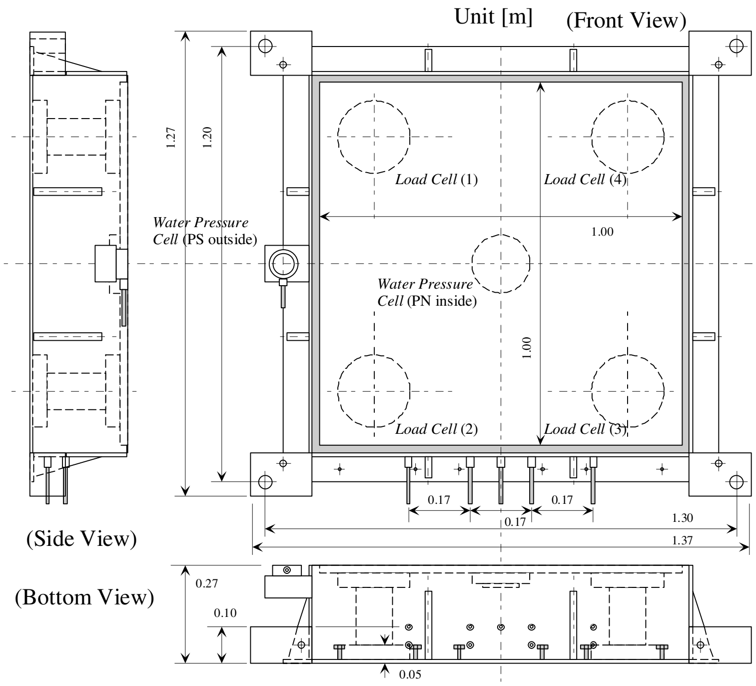

Fig. 12. Earth pressure cell with large sensitive panel

On the front wall of the test caisson in the treated area, four wave pressure cells (TW1, 2, 3, 4) were vertically arranged, to measure not only the change in water pressure induced by wave itself but also the dynamic change induced by earthquakes and vibrations of the caisson.

On the back of the test caisson four earth pressure cells were vertically arranged both in the treated and untreated areas (TN1, 2, 3, 4 and UN1, 2, 3, 4). The details of the earth pressure cell are shown in Fig. 12. The large sensitive panel of 1.0 m x 1.0 m, which was coated with cement mortar to guarantee similar frictional properties with the caisson back, was supported by four load cells. Each of the earth pressure cells have two water pressure cells both inside and outside the sensitive panel to monitor dynamic change in water pressure; this makes it possible to calculate effective and total earth pressure with the correction with respect to the inertial force on the sensitive panel and the inside water pressure. In the top third earth pressure cell (TT3, UT3), not only normal component but also tangential component of the earth pressure can be measured with the additional strain gauges on the load cells.

Four vertical holes in total were dug for the measurement of permanent displacement of the test caissons and backfill ground induced by large earthquake by means of an inclinometer.

Data Acquisition

The data acquisition system is shown schematically in Fig. 13. All the sensors are grouped into eleven units in the system, and electric signal is converted from analogue to digital. The supervision and data collection would be automatically conducted by the main computer set at the observatory house set up beside the test quay wall. The obtained data can be sent by remote control to the head quarter of this project in Civil Engineering Research Institute of Hokkaido through ISDN data communication system.
Four types of measuring scheme are available in this project:

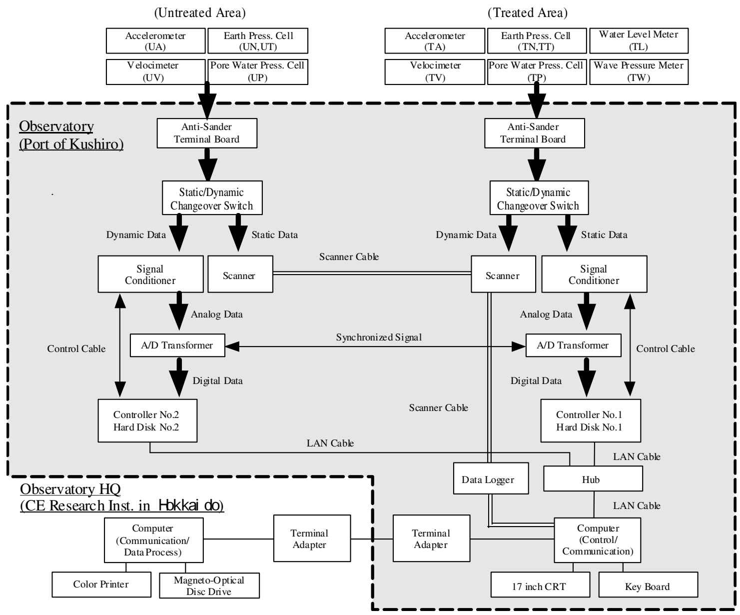

Fig. 13. Data acquisition system.

Correction measurement is automatically carried out at 12:30 everyday. Sensitivity of all the sensors except accelerometers and velocimeters is monitored, and the obtained data is used for correcting the data from Earthquake measurement.

Static measurement is also automatically carried out four times every day. The static response of the test quay wall is monitored in this measurement, and the obtained data is can be combined to Earthquake measurement to calculate absolute values from relative values.

Earthquake measurement is a dynamic measurement with a frequency of 100Hz carried out during earthquake. This measurement is triggered by the accelerometer TA8 at base layer AS2. If the absolute acceleration calculated from vertical component (TA8-z) exceeds the threshold value of 1Gal or 0.5Gal, the data for 181s from 20s before the trigger are recorded in the storage in a computer.

Extended earthquake measurement is carried out following Earthquake measurement in the case of large earthquake; and the data from all the sensors except accelerometers and velocimetes are stored. If large earthquake occurs and the seismic intensity recorded at surface in vertical direction (TA5-z) exceeds 3.5, this measurement is started and the data from all the sensors except accelerometers and velocimeters are recorded for following 1800s (30min). In this measurement the dissipation of pore water pressure and associated static change in earth pressure can be traced.

OBSERVED BEHAVIOR IN 2003 TOKACHI-OKI EARTHQUAKE

In this section the vibration behavior of test quay wall observed in 2003 Tokachi-oki Earthquake is briefly presented, which shook the observation site with seismic intensity of rank V+. Figure 14 shows the time histories of acceleration recorded at the surface of backfill ground, top of the caissons and in the seabed. As shown in this figure, the maximum acceleration exceeded 100Gal at ground surface, and the characteristic spikes suggesting the occurrence of liquefaction of the backfill ground can be recognized at ground surface (TA5, UA5); the spikes were clearer in the untreated area compared with the treated area.

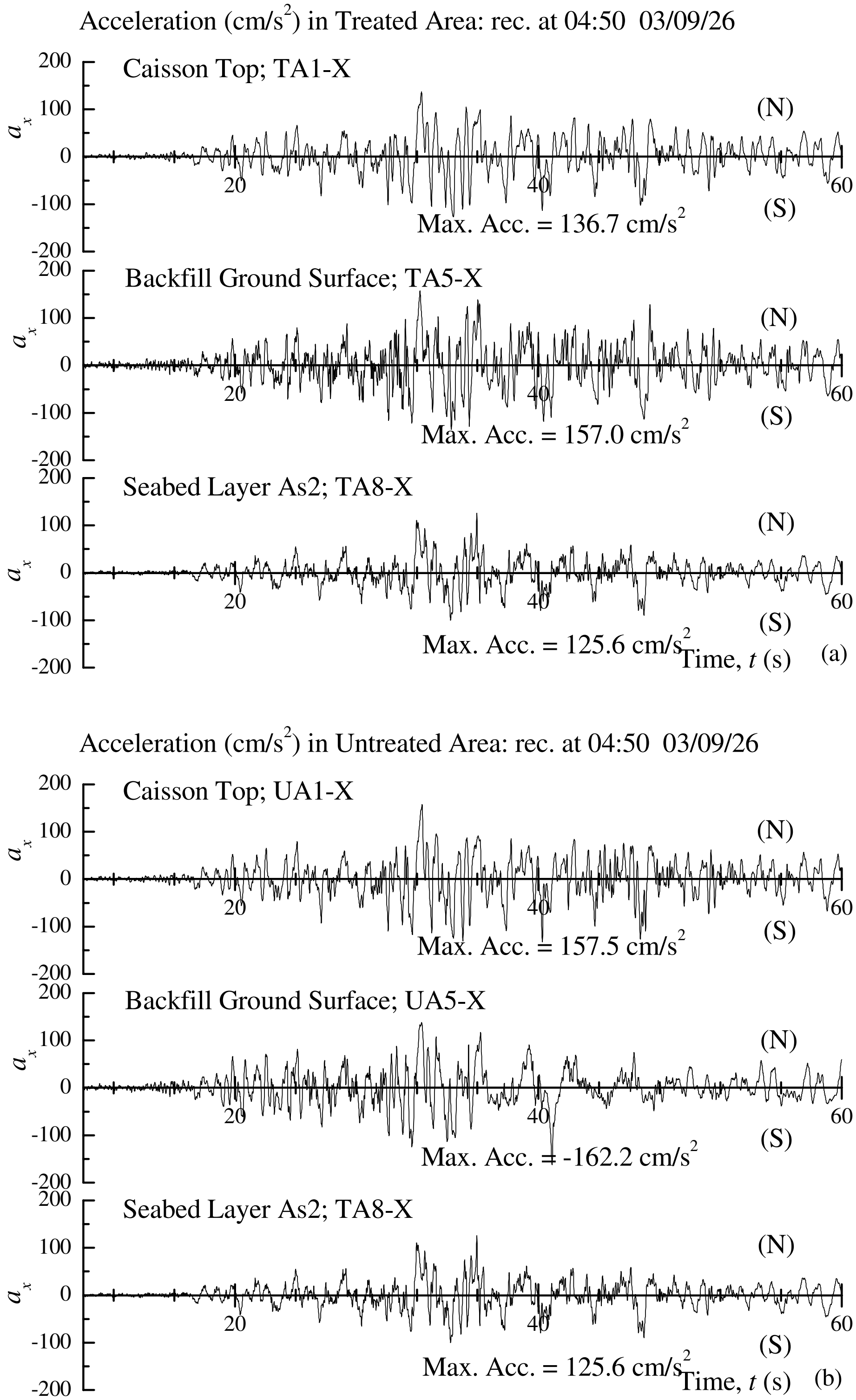

Fig. 14. Time history of acceleration in N-S direction;
(a) Treated area, (b) Untreated area

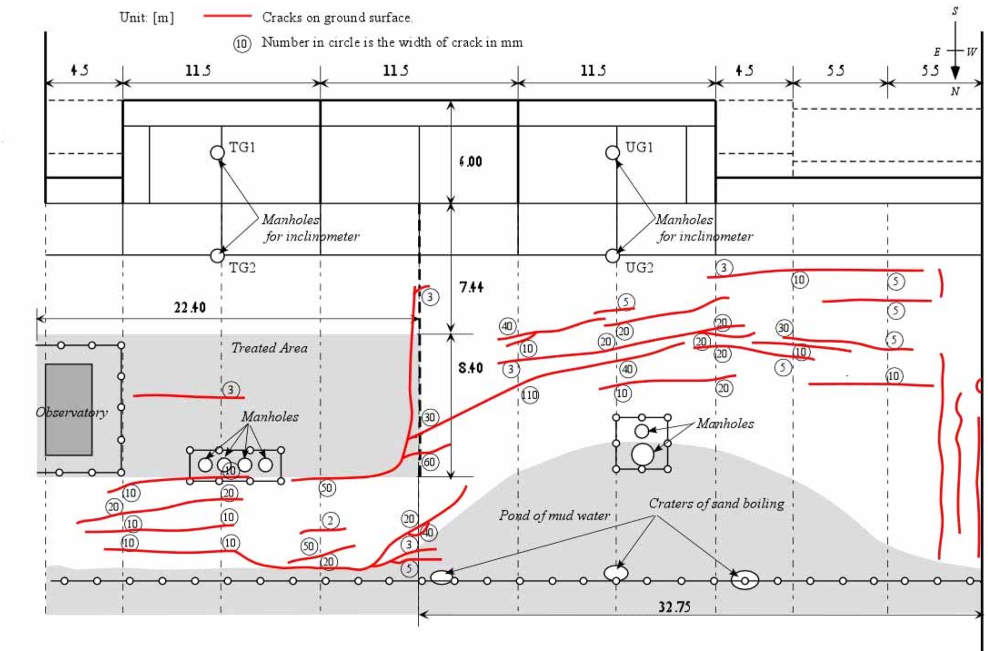

Fig. 15. Deformation of the backfill ground with cracks, boiled sand and muddy water

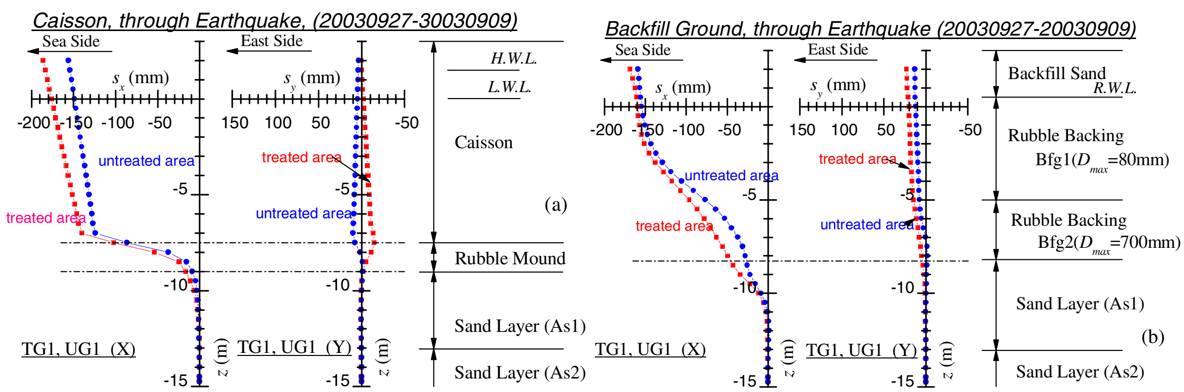

Fig. 16. Vertical distribution of displacement by using inclinometer;
(a) at caisson, (b) at backfill ground

Shown in Fig. 15 is the deformation of ground surface; the surface cracks appeared over the backfill ground surface except for the treated area. Some craters for sand boiling also appeared and pond of the muddy water was seen for a few days after the earthquake. The displacements of the caissons were approximately 20 cm according to the survey by means of GPS, and clear difference could not recognized between in the treated and untreated areas. The vertical distribution of displacements of the caisson and the backfill ground were measured by means of inclinometer both in the treated and untreated areas; see Fig. 16. The displacement distribution suggests the occurrence of slip at the caisson bottom and/or shear deformation of rubble mound.

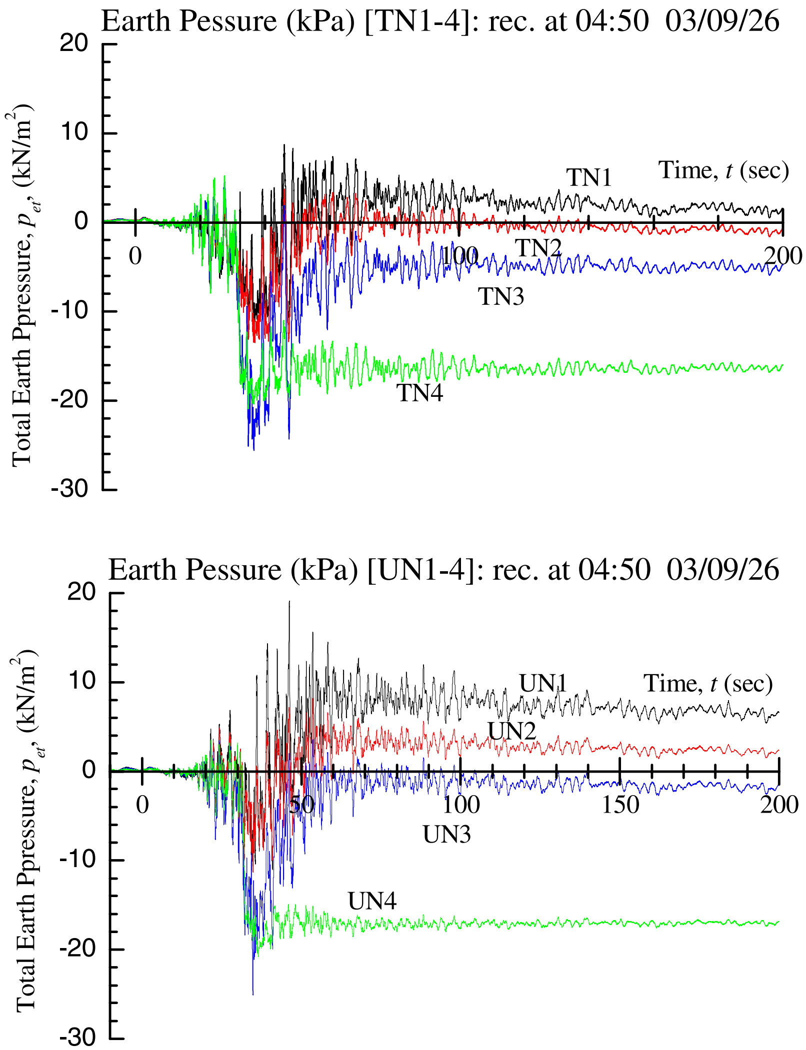

Fig. 17. Change in total earth pressure during shaking;
(a) Treated area, (b) untreated area

The change in total earth pressure on the back of the caissons is shown in Fig. 17. The notable decrease in the total earth pressure can be seen during shaking. This decrease was not permanent and followed by the increase in the total earth pressure after shaking. The instantaneous decrease in the total earth pressure was clearer at deeper earth pressure cells.

CONCLUDING REMARKS

The outline of a project for observing the dynamic response of quay wall to earthquake was introduced in this paper. First, the background and the motivation of the project were explained. Next, the selected site, the West District of Port of Kushiro, eastern Hokkaido, Japan, was introduced. The details of the test quay wall were explained; the construction sequence, the employed remedial measure with sand compaction pile against liquefaction, the geological condition, and the system for data acquisition were described.

Tokachi-oki Earthquake which occurred on 26 September 2003 shook the Port of Kushiro with seismic intensity of V+. The maximum acceleration exceeding 100 cm/s^2 were recorded at the surface of backfill ground, and the caissons were permanently displaced toward the sea by about 20 cm. The vibration behavior of the test quay wall in the earthquake was briefly introduced in this paper. The authors are now analyzing the observed behavior of test quay wall, and will present some other papers on the analytical results elsewhere in the near future. They are planning a new cooperative research project with other researchers in Japan and other countries on the seismic stability of water front structures. In the new project the behavior of the test quay wall observed in this project will be opened for the detailed analyses by the members the project.

Acknowledgement
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