MODEL TESTS ON BEHAVIOUR OF GRAVITY-TYPE QUAY WALLS SUBJECTED TO STRONG SHAKING

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SUMMARY

Seismic stability of gravity-type quay walls and prevention of their large distortion are of major concern from a disaster prevention view point as well as in the sense of successful restoration after strong seismic events. There are, however, many existing walls which are of limited seismic resistance and would not be safe under increasing magnitude of design earthquakes. The present study conducted shaking model tests in both 1-g and 50-g centrifugal fields in order to demonstrate the efficiency of available mitigation technologies. Test results suggest that soil improvement in the loose foundation sand can reduce the quay wall damage to a certain extent when the intensity of shaking is around 0.30g. In contrast, under stronger shaking, the centrifugal tests manifested that those measures are not promising because of the increased effects of seismic inertia force.

INTRODUCTION

Among many kinds of liquefaction-induced damages during earthquakes, those of harbour quay walls attract special concern. This is because transportation to and from harbours often plays key roles in evacuation and emergency rescues as well as in reconstruction of the damaged area. Moreover, harbours are often economic centres of a region and their malfunctioning may affect the local economy after a quake.

A caisson quay wall is a gravity type quay wall. Damages to caisson quay walls during the 1995 Kobe earthquake have shown the vulnerability to earthquakes of this type of structures [1], especially when the foundation was a replaced sandy soil. To understand the behaviour of this type of structures during earthquakes, model test studies have been done by many researchers (e.g. [1], [2], [3]). However, only a few studies (e.g. [4], [5]) have been reported about the damage mitigation systems for existing gravity type caisson quay walls. This situation demands more studies on the effectiveness of mitigation systems for caisson quay walls.

Figure 1 illustrates the significant distortion of Kobe Harbour in the 1995 earthquake. This part of the harbour consisted of a gravity quay wall with loose backfill sand behind. Such a distortion stopped the function of the harbour and affected the local economy for a long time. Figure 2 indicates another effect of the distortion of quay walls; the backfill area was subjected to significant tensile deformation as evidenced by surface cracks. This kind of ground distortion would affect embedded structures and destroy them. In this context, seismic reinforcement of existing harbour quay walls is frequently discussed.

One significant problem in the retrofitting of existing structures is the increasing magnitude of design earthquakes. This situation is partly due to the increase in the maximum accelerations which have been recorded during recent earthquakes (Table 1 and Figure 3). Another reason is certainly the awareness of the economic and social effects due to damage of important structures. It is noteworthy that the recent discussion on seismic design under very strong earthquake motion allows a certain extent of deformation, while total collapse has to be prevented. This idea appears to be valid for gravity quay walls as well. With these issues in mind, the present paper addresses model tests in both 1-g and centrifugal field that were conducted on mitigation of large deformation of gravity quay walls. The key issues therein were retrofitting of “existing structures” and “very strong earthquake shaking”.

FAILURE MODES OF QUAY WALLS

Based on previous 1-g shaking table model tests and damage surveys of past earthquakes, possible failure modes of quay walls during earthquakes are identified as follows:

1. Sliding of the quay wall on rubble mound with a failure wedge in the rubble filter and backfill.
2. Sliding of the quay wall, rubble mound and rubble filter on foundation soil.
3. Shear deformation in the foundation soil and loss of soil from foundation under the toe of the quay wall, causing horizontal and vertical displacements, and seaward tilting of the quay wall.

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4. Formation of circular slip plane passing through the foundation soil, causing horizontal and vertical displacements, and landward tilting of the quay wall.

The main forces acting upon the quay wall and foundation during dynamic loading are as follows:

1. Relative inertia force
2. Static earth pressure on the back face of the quay wall
3. Dynamic earth pressure on the back face of the quay wall
4. Static shear stresses in the foundation soil
5. Non-uniform vertical stresses on the foundation

Three modes of seismic response of the quay wall were identified during dynamic loading, namely:

1. Horizontal or translational shaking
2. Vertical motion
3. Rotational shaking or rocking

In the model tests presented in this paper, the base excitation had only a horizontal component. Vertical vibration was not considered.

1-g SHAKING TABLE TESTS

Shaking table tests were performed using dry model ground and saturated model ground. Dry model ground was used to investigate the behaviour of quay walls without effects of soil liquefaction. Saturated model ground was used to observe the behaviour of quay walls subjected to seismic liquefaction in the foundation soil and backfills. Figure 4 indicates a typical cross section of the gravity wall which was studied in the present investigation.

Table 1: Maximum horizontal ground accelerations of some earthquakes in the last century and recent decades.

Year	Earthquake	Acc (g)		Year	Earthquake	Acc (g)
1891	Nobi, Japan	0.43		1970	Hidaka Sankei, Japan	0.44
1923	Kanto, Japan	0.55		1974	Izu Hanto Oki, Japan	0.44
1927	Kita Tango, Japan	0.50		1978	Izu Hanto Kinkai, Japan	0.45
1930	Kita Izu, Japan	0.54		1978	Miyagi Ken Oki, Japan	0.29
1933	Long Beach, USA	0.23		1983	Nihonkai Chubu, Japan	0.40
1933	Noto Hanto, Japan	0.43		1987	Chiba-toho oki, Japan	0.26
1939	Oga, Japan	0.43		1989	Loma Prieta, California, USA	0.28
1940	Imperial Valley, USA	0.33		1993	Kushiro Oki (Weather Stn), Japan	0.92
1943	Tottori, Japan	0.57		1993	Kushiro Oki (Kushiro Port), Japan	0.31
1946	Nankai, Japan	0.54		1993	Hokkaido Nansei Oki, Japan	0.22
1948	Fukui, Japan	0.64		1994	Sanriku Haruka Oki, Japan	0.60
1949	Imaichi, Japan	0.36		1994	Northridge, USA	1.74
1955	Futatsui, Japan	0.40		1995	Kobe (Marine obsevatory), Japan	0.82
1962	Hiroo Oki, Japan	0.34		1995	Kobe (Kobe University), Japan	0.31
1962	Miyagi Hokubu, Japan	0.47		1995	Kobe (Port Island), Japan	0.34
1964	Niigata, Japan	0.17		1999	Adapazari, Turkey	0.40
1966	Matsushiro, Japan	0.54		1999	ChiChi, Tiwan	1.00
1966	Park Field, California, USA	0.48		2000	Hino, Tottori, Japan	0.63
1967	Bombay, India	0.63		2002	Bam, Iran	0.80
1968	Ebino, Japan	0.40		2003	Tokachi Oki, Japan	0.80
1968	Tokachi Oki, Japan	0.24		2004	Ojiya, Japan	1.31

Table 2: Caisson displacements and backfill subsidence during shake1, shake2 and shake3 of model test QW-01dry.

	Base motion characteristics	Horizontal displacement (mm)	Vertical displacement (mm)	Seaward tilting of wall (degrees)	Subsidence of backfill surface (mm)
Shake1	0.265g/10Hz/40*	2 (approx.)	< 1.0	< 0.2	20
Shake2	0.265g/10Hz/40	1	< 0.5	<0.1	2
Shake3	0.535g/10Hz/40	80	14	3.5	Not uniform

*0.265g/10Hz/40 means the maximum acceleration amplitude is 0.265g, the frequency is 10 Hz and the number of cycles with amplitudes greater than 0.05g is 40.

Figure 1: ***Significant distortion behind a gravity quay wall in Kobe Harbour.***

Figure 2: ***Failure of backfill ground behind gravity quay wall (Nishinomiya Harbour).***

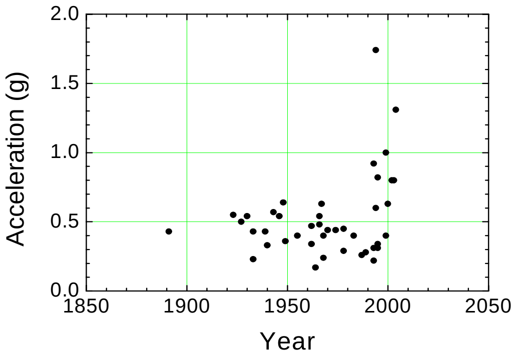

Figure 3: ***Increasing values of recorded maximum accelerations in recent decades.***

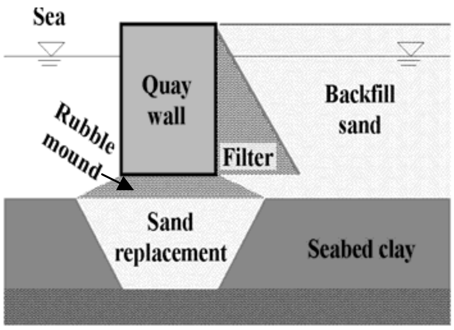

Figure 4: ***Conceptual cross section of investigated gravity quay wall***

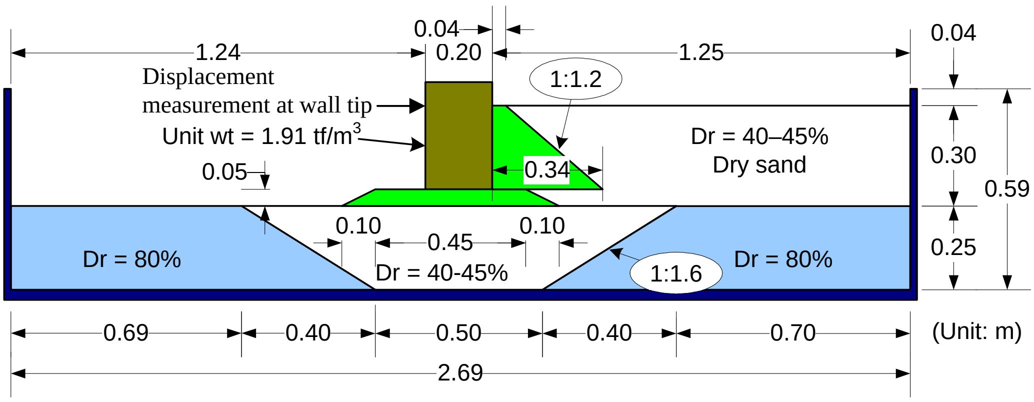

Figure 5: ***Cross section of model QW-01dry.***

Dry Model Ground

A quay wall model test QW-01dry was performed with dry Toyoura sand as backfill and foundation. The model ground and wall were prepared in a rigid container 2.69 m long by 0.39 m wide and 0.59 m deep on a shaking table of 3.0 m by 2.0 m. Cross section of the model is shown in Figure 5. The compacted dense sand layer was prepared at the bottom by wet tamping of Toyoura sand in layers and the foundation and backfill were prepared by placing air dry Toyoura sand with the help of a scoop. So it is obvious that density of the foundation and backfill sand may not be uniform. However the trials of the same method outside of the model container yielded a 40 – 45% relative density of Toyoura sand.

This dry quay wall model called QW-01dry was shaken three times (shake1, shake2 and shake3) at 10 Hz frequency. Schofield and Scott [6] suggested that frequency has to be increased by a factor of $n^{0.5}$ for a 1/n scaled down model. This was the reason for applying base motions with frequency 10 Hz ($\cong 1.5 \times 50^{0.5}$) whereas the assumed prototype frequency is 1.5 Hz. Shake1 and shake2 had a maximum acceleration amplitude of 0.265g while shake3 had a maximum acceleration amplitude of 0.535g. The caisson displacement and backfill subsidence during three shakings are listed in Table 2. During shake1 and shake2, there was densification of backfill and foundation. However, horizontal and vertical displacements of the quay wall were negligible during shake1 and shake2. There was no evidence of failure wedge in the backfill during shake1 and shake2 with acceleration amplitude of 0.265g. However, during shake3 with increased acceleration amplitude of 0.535g, the quay wall moved seaward significantly as much as 80 mm measured at the top of the quay wall and vertical displacement at the front of the quay wall was 14 mm with seaward tilting of 3.5 degrees.

The base motion and quay wall displacements during shake3 are shown in Figure 6. When acceleration amplitude exceeded 0.4g, quay wall displacement and tilting started, which indicated that the designed caisson wall is stable against an acceleration level below 0.4g. Simple stability calculations against earthquake induced inertia force of quay walls was done on the basis of the force diagram in Figure 7. When the friction angle between the rubble mound (gravel) and caisson (wooden box) is assumed to be 35^o [7], 0.40g acceleration induces a 500 N inertia force per meter width of the quay wall which can not mobilize the translation of quay wall when only static active earth pressure is mobilized upon small displacement of quay wall. Because frictional resistance under the quay wall was 660 N forces per meter width and active thrust of the backfill was 75 N forces per meter width. Therefore, it is reasonable to say that additional required force for mobilizing translation of quay wall came from the dynamic earth pressure of the rubble filter. The photograph shown in Figure 8 was taken after application of shake3 with acceleration amplitude 0.535g which shows a failure wedge that developed during the strong shaking.

It was also observed that the vertical displacement of the quay wall during shake3 with acceleration amplitude 0.535g was not due to densification of the foundation sand but mainly due to distortion of the rubble mound which was placed loosely, tamped lightly and levelled before placing the quay wall. It should be noted that the rubble filter was also placed loosely and tamped lightly in all model tests. Visual observation of the deformed backfill and rubble filter revealed that a failure plane or shear band under the active soil wedge was oriented towards the direction of around $50\text{-}55^o$ from the horizontal.

Results of test QW-01dry indicate that maximum acceleration of 0.265g was not enough to produce the required inertia force of the quay wall and dynamic earth pressure for sliding of the quay wall to occur while the acceleration of 0.535g was large enough for that. This kind of displacement may be predicted by the Newmark sliding block method [8]. One outcome is very clear from this experiment. Namely, for a dry foundation and backfill, the caisson wall can be easily designed for a certain level of earthquake loading so that it remains intact during earthquakes of intensity less than that level. Above the design level of the earthquake, the quay wall displacement becomes significant during shaking. In this model test, there was no liquefaction in the backfill and foundation sand because the sand was dry. Damage of the quay wall still occurred, however, when the shaking amplitude was increased to 0.535g. Sliding on a rubble mound and local distortion of the rubble mound at the toe of the quay wall were responsible for horizontal, vertical and angular displacements of the quay wall at this increased level of shaking. It means that in the case of a very strong shaking, a quay wall system might be damaged even without generation of excess pore pressures in the backfill and foundation soils. On the other hand, if the earthquake motion is less than the design level of the earthquake and at the same time the backfill and foundation sands do not liquefy, a quay wall system would suffer no damage as was observed during shake1 and shake2 with acceleration amplitude of 0.265g in the model test QW-01dry.

Saturated Model Ground

In real field conditions where seabed soil consists of soft marine clay and a heavy gravity wall needs to be placed upon such soils, slope instability and time dependent consolidation settlement of the gravity quay wall have to be avoided. Hence, the soft clay is often replaced by granular materials. This type of caisson quay wall was modelled at a scale of 1/50 of a typical prototype in Port Island and Rokko Island of Kobe, Japan. The cross section is illustrated in Figure 9. Toyoura sand of different relative densities (Figure 9) were used to simulate the prototype quay wall site. The marine unliquefiable clay layer at the quay wall site was modelled by dense (Dr = 80%) Toyoura sand to avoid the difficult and time consuming handling of an unliquefiable clay in ground preparation. From a liquefaction view point, dense Toyoura sand is equivalent to marine clay. However, the relative friction angles between dense sand and other parts of the model are not the same as that between soft clay and the other components. This is a limitation of the adopted model.

Confining pressure strongly affects the undrained behaviour of sand [9]. To reproduce the field behaviour in 1-g model under very low effective stress, the backfill and foundation sands were prepared at a low relative density of 30-35%.

Model Preparation

A layer of 25 cm thick Toyoura sand with relative density of 80% was prepared layer by layer by the wet tamping method with 2.5 cm layer thickness. Soil under the quay wall was excavated maintaining a desired slope of 1:1.5 on both sides. Carbon dioxide (CO_2) was percolated from the bottom of the container to remove the pore air. Then, water was supplied from the bottom of the container at a slow rate until the water level reached the sand surface. The excavated trapezoidal channel was filled by raining sand uniformly from the top of the model container. This method of ground preparation is called water sedimentation. In this method, sand particles settle at their terminal velocities. Repeatable and uniform sand deposit can be prepared by this method. Moreover, no extra effort is necessary for saturation. In construction of quay walls in the field, the foundation sand is prepared under sea water as well, which is another reason for adopting the water sedimentation method in the present model tests.

Sand raining was stopped temporarily to place coloured sand targets, accelerometers and pore pressure transducers. When the trapezoidal channel was filled up, the water level was increased to 30 cm from the bottom. At this stage, the rubble mound was prepared by placing crushed stone chips with an average particle size of 3.5 mm and dry density of 1400 kg/m^3. The wall was placed directly on the rubble mound and the water level was further raised up to a height of 45 cm. The sand was rained on the backfill side up to a height of 30 cm and a stone filter was prepared using the same crushed stone chips. Cut pieces of 0.4 mm thick geotextile were installed as an interface between the crushed stone and sand backfill. The rest of the backfill was filled by sand raining. The water level was kept at 1.0 cm above the model ground surface.

Weight of quay wall

The effective height of the quay wall was 25 cm and the standard width-to-height (aspect) ratio of 0.80 was adopted. The weight of the wall was designed assuming that a uniform effective contact pressure under the wall would be 10% more than the effective stress at 25 cm depth of sand backfill. The centre of gravity of the wall was located at 12.3 cm above the bottom of the caisson box.

Shock absorber

To minimize the P-wave generation from both ends of the rigid model container, a dry mattress made of polymer wires encapsulated with impermeable polythene sheet was used as

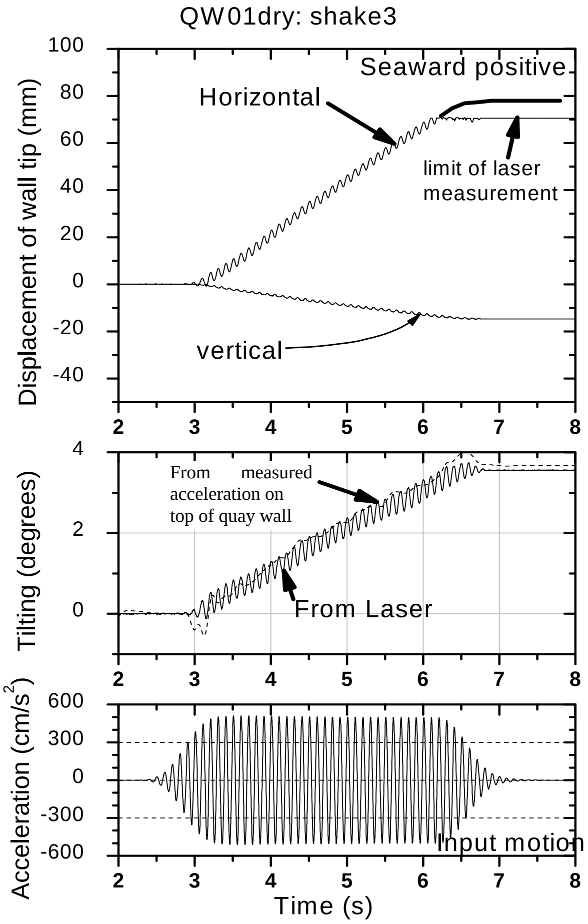

Figure 6: ***Wall displacement time histories and input motion of shake3 of test QW-01dry.***

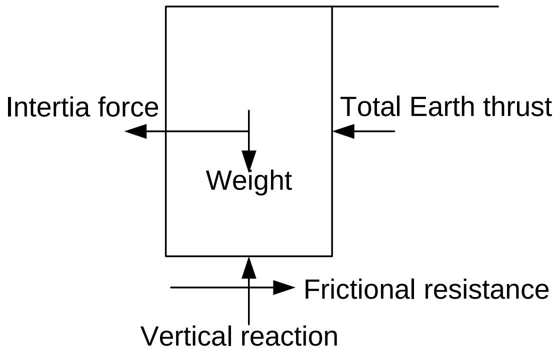

Figure 7: ***Components of forces working on a quay wall during dynamic loading (Simplified).***

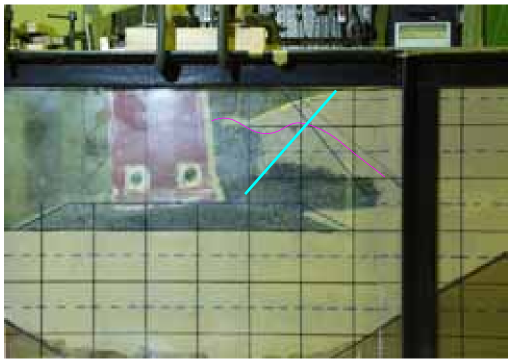

Figure 8: ***Displacement of caisson and deformation of backfill of test QW-01dry after shake3.***

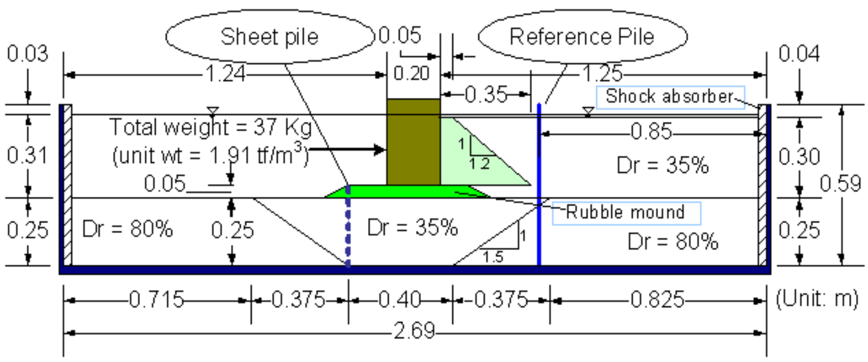

Figure 9: ***Configuration of model ground made of Toyoura sand employed in 1-g shaking table tests.***

an interface between the sand and the end walls. Because the friction was very small between the wall and sand after liquefaction, no lubricating agent was used in the side walls.

Results

Figure 10 illustrates one of the past tests which demonstrate a typical shape of the displaced quay wall. It may be seen that the quay wall translated towards the sea (left) and tilted forward as well. This wall movement was associated with the lateral displacement of the backfill soil together with the foundation sand beneath the wall. This situation is simplified in Figure 11.

Models thus prepared had three kinds of mitigation measures installed. The first one was densification of the sand at the sea side of foundation (Figure 12). The second was an installation of a 2 mm aluminium wall at the sea side of the foundation sand (Figure 13); it was intended to reproduce the effects of a sheet pile wall that would restrain the lateral displacement of the foundation sand. Similarly, a sheet pile was installed in the backfill (Figure 14) by which it was intended to stabilize the backfill soil and reduce the dynamic earth pressure acting on

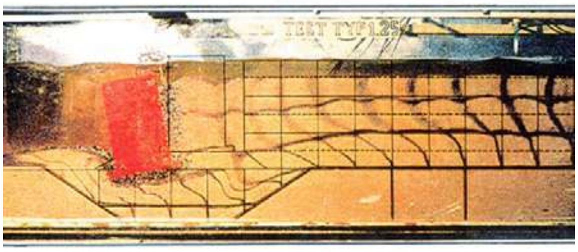

Figure 10: Displacement of quay wall and distortion of model ground in a 1-g model test (Ghalandarzadeh et al., 1998).

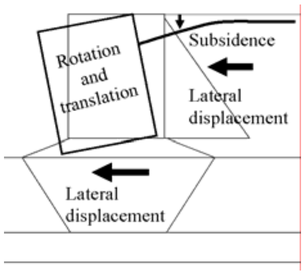

Figure 11: Typical configuration of quay wall distortion.

the quay wall. A further attempt was made to execute permeation grouting in the foundation (Figure 15) in order to reduce the distortion of the foundation sand. The employed base excitation in this test series is illustrated in Figure 16 in which the amplitude of acceleration was as large as 0.30g with a frequency of 10 Hz.

The recorded time history of the lateral displacement at the top of the quay wall is plotted in Figure 17. It can be seen that the displacement obtained from the cases of a sheet pile wall in the backfill soil was as large as that in the case without mitigation. This implies that sheet pile mitigation behind a quay wall is not effective. A possible reason for this is that the dynamic earth pressure exerted by the backfill soil was not reduced by a sheet pile installed far behind the quay wall; there remained a big mass of soil between the sheet pile and the quay wall which generated a substantial magnitude of inertia force. Note that the installation of a sheet pile wall at a shorter distance is not practical because there is a filter zone made of large stones. Another reason is the overwhelming role played by the inertial force of the caisson wall. Evidently, this force was not mitigated by the sheet pile wall in the backfill, and hence, a significant wall distortion resulted.

Figure 18 shows the time history of rotation of the quay wall for the different cases. It is seen that the rotation was maximal when no mitigation was taken, and that the installation of a sheet pile wall behind the quay wall was not effective. In contrast, the rotation was reduced by mitigation measures conducted in the foundation under the quay wall or in the sea side of the foundation. The observations in terms of lateral displacement (Figure 17) and rotation (Figure 18) are consistent. So grouting in the foundation soil, sheet pile in the sea side of the foundation, and densification in the sea side of the foundation were found to be effective for mitigation of the damage of quay wall systems.

50-g CENTRIFUGE TESTS

The study was continued to model tests under 50-g centrifugal gravity field. The rate of pore pressure dissipation was adjusted by employing silicon oil as pore fluid that had 50 times more viscosity than water. All the data are presented here in the prototype scale. Scaling factors are listed in [10]. As the similitude rules were used to convert the data from a prototype scale to a model scale, the wall height to wavelength relationship was the same for the centrifuge model and the quay wall in the field. The shaking acceleration level was equivalent to 0.43g at 2 Hz for 15 seconds in the prototype scale. This level of acceleration with a substantial number of cycles (Figure 19) seems to be a very strong motion. Tested models were constructed by using Toyoura sand which was of around 40% relative density (Figure 20).

Figure 21 shows the side view of the model without mitigation before the test. Figure 22 shows the same model after the shaking. The displacement of the soil was measured by both displacement transducers at relevant locations and by taking pictures of coloured sand markers embedded in the model. Figure 23 was produced by superimposing Figure 21 and Figure 22 to clearly show the distortion of the model caused by the shaking.

The studied mitigation methods were a sheet pile on the sea side and grouting in the foundation sand under the quay wall. For a comparison purpose, moreover, another model test was performed using dense (Dr = 65%) Toyoura sand for both the foundation soil and backfill. This model is called hereafter as overall densification.

Detailed information on lateral translation and tilting of a quay wall is presented in Figures 24 and 25. When no mitigation measure was taken (Figure 23), the quay wall model simply translated and subsided downwards, while tilting was very small. When the foundation soil was improved by grouting, on the other hand (Figure 25), more significant extent of tilting in the wall model occurred. Both tests developed significant subsidence in the surface of the backfill. As was suggested in previous figures, the case without mitigation developed smaller lateral displacement at the top of the quay wall, and the rotation was minimal in the case without mitigation. This unexpected finding may be interpreted by dividing the lateral displacement into components. Figure 26 shows shear strains in the foundation sand and in the rubble mound which were read from the displacement of embedded markers. The case without mitigation developed the greatest extent of shear deformation in the foundation sand, while the rubble mound exhibited less strain and rotation of the quay wall was negligible. On the other hand, cases with reinforced or solidified foundation developed more tilting of the quay wall while the strain in the foundation was smaller. The stabilized foundation soil increased the acceleration in the overlying structures and in consequence magnified the distortion of the rubble mound, Moreover, the inertial force, translation, and tilting of the quay wall were increased. Accordingly, the dynamic earth pressure exerted from the backfill became more significant.

From the displacement of embedded markers (Figure 21 and Figure 22) shear strain in the foundation was calculated. Detailed distribution of shear strain, ε_1- ε_3, in the foundation sand is illustrated in Figure 27 through Figure 30. The case without mitigation (Figure 27) developed significant strain because the sand therein was very soft. Figure 28 shows the case of a sheet pile wall. Since the sand moved towards the space between the sheet pile and the quay wall, the strain is not small. In contrast to these two cases, grouting and overall densification (Figure 29 and Figure 30) reduced the magnitude of shear strain.

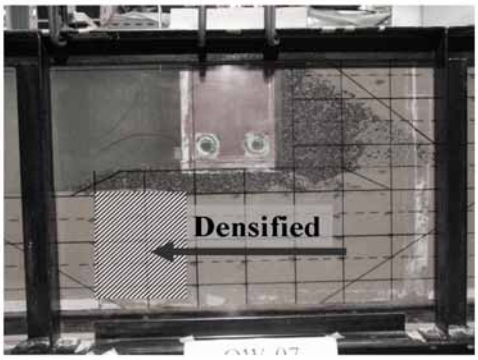

Figure 12: Densification in sea side of foundation.

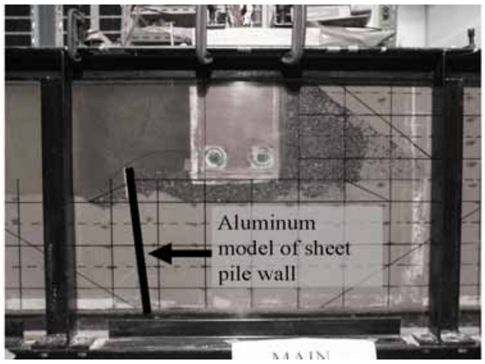

Figure 13: Sheet pile in sea side of foundation.

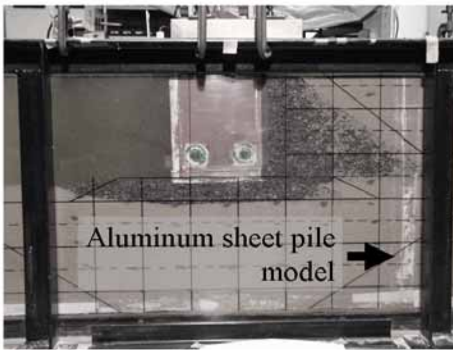

Figure 14: Sheet pile in backfill

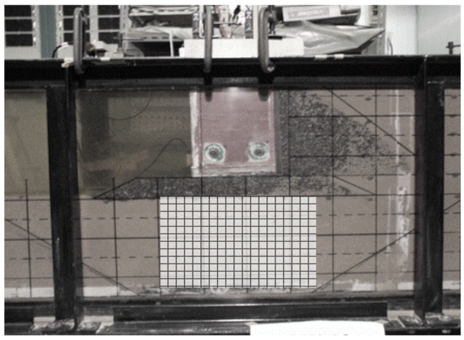

Figure 15: Grouting in foundation.

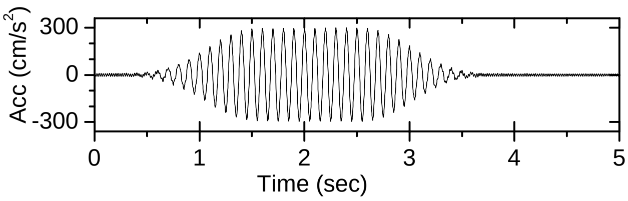

Figure 16: Time history of base shaking.

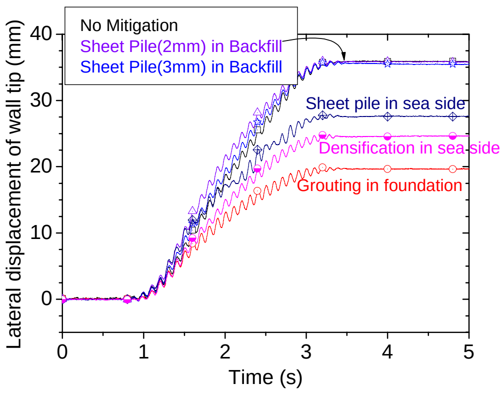

Figure 17: Time history of lateral displacement at top of quay wall.

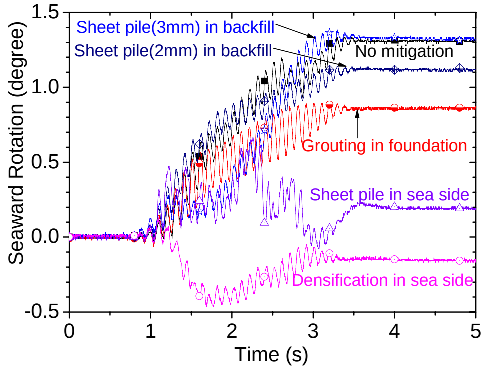

Figure 18: Time history of rotation of quay wall.

DISCUSSION

The present study addresses the behaviour of a gravity quay wall subjected to strong shaking. Displacement of the quay wall and distortion in the backfill in saturated 1-g model tests were mainly due to liquefaction. The quay wall model with dry foundation and backfill suffered no damage under nearly the same level of shaking. In the case of liquefaction and no mitigation measures taken, conversely, the significant extent of distortion was evident in both 1-g and centrifuge tests in the

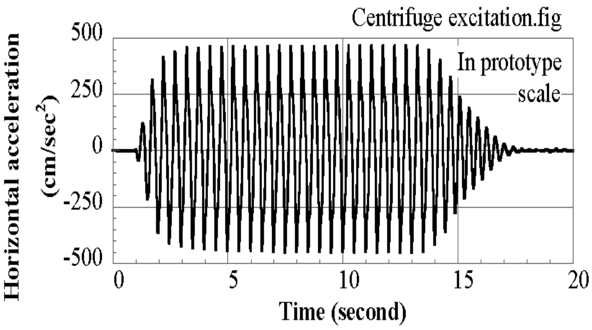

Figure 19: Time history of base excitation (prototype scale).

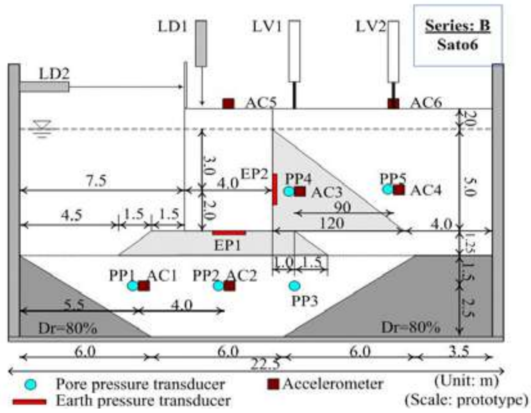

Figure 20: Configuration of model ground employed in 50g gravity field.

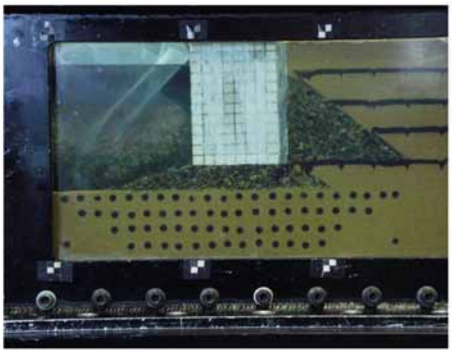

Figure 21: Appearance of centrifugal model of gravity quay wall before shaking (no mitigation case).

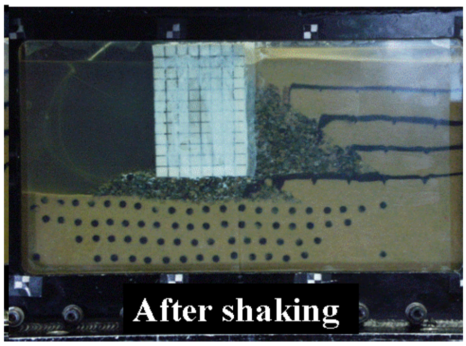

Figure 22: Appearance of quay wall model after shaking (no mitigation case)

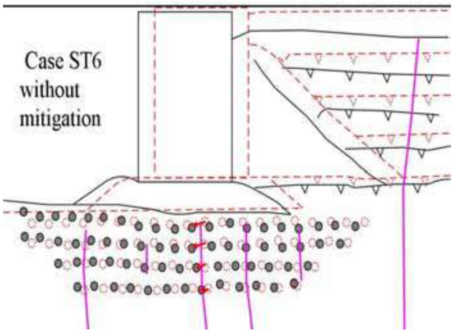

Figure 23: Distortion and strain information collected from photographs before and after shaking.

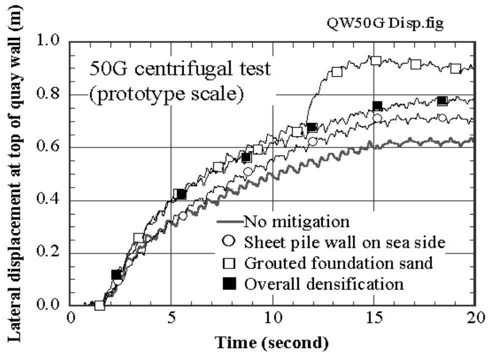

Figure 24: Time history of lateral displacement at top of quay wall.

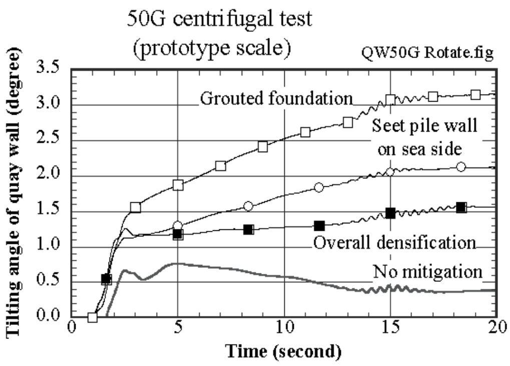

Figure 25: Time history of tilting angle of quay wall.

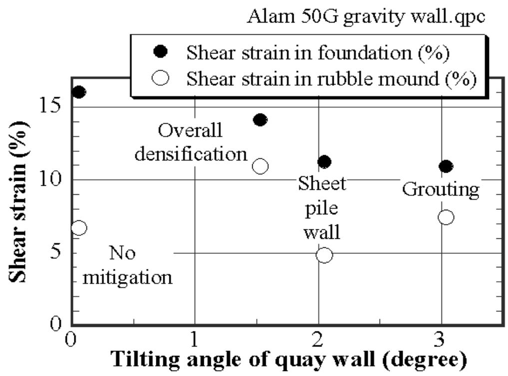

Figure 26: Shear deformation in foundation and rubble mound.

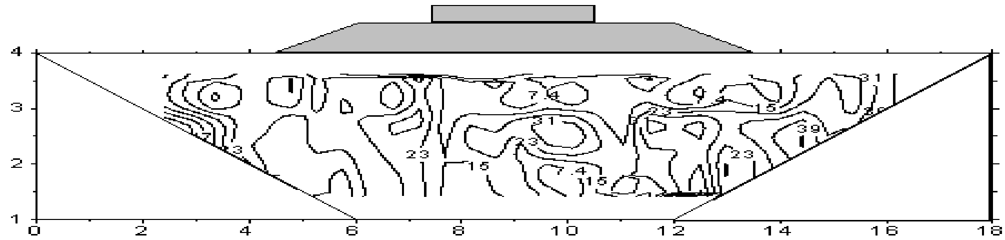

Figure 27: Distribution of shear strain in the model without mitigation.

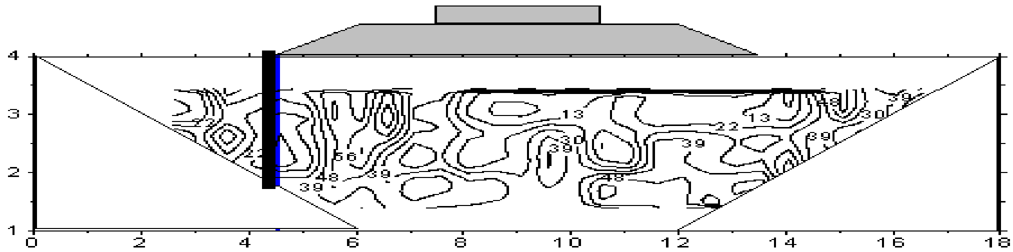

Figure 28: Distribution of shear strain in the model with sheet pile wall in sea side of foundation.

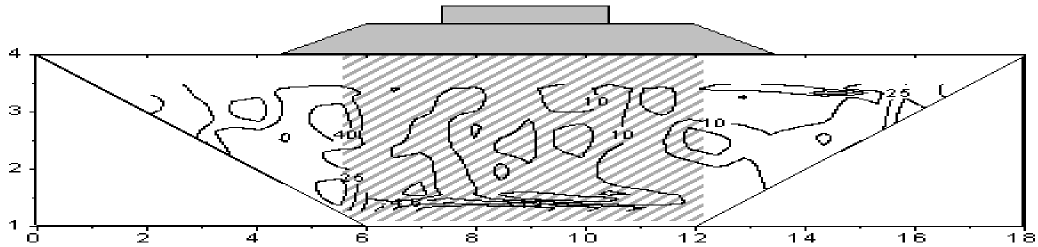

Figure 29: Distribution of shear strain in case of grouting in foundation.

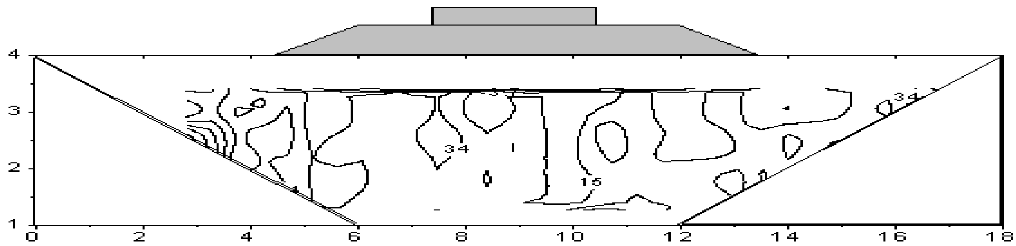

Figure 30: Distribution of shear strain in case of overall densification.

models with no mitigation. The effects of mitigation were observed in 1-g tests as expected. However, the mitigation effect was not clear or even opposite in centrifuge tests.

The reason for the inverse finding in centrifuge tests is not clear, but some discussion may be possible. The centrifuge tests employed greater magnitude of base shaking (0.43g vs. 0.30g in 1-g tests). This strong shaking developed significant inertial forces in the wall and the backfill. The backfill consequently exerted substantial dynamic earth pressure on the quay wall. Since the bottom of the wall was rather stable due to densification, solidification, or constraint by the embedded sheet pile wall (see solid circles in Figure 26), the quay wall tended to rotate rather than to translate. This rotation led to significant lateral displacement at the top of the quay wall. It seems consequently that the present tests may suggest the limited stability of gravity-type quay walls subjected to very strong earthquake shaking.

CONCLUSION

Shaking model tests were carried out on mitigation of seismic distortion of a gravity-type quay wall. When the shaking was conducted with the magnitude of 0.30g in a 1-g field, such mitigation measures as grouting in foundation, densification of sand in the sea side of the foundation and sheet pile in the sea side of the foundation were able to reduce the distortion of quay walls. It was noteworthy, however, that a sheet pile wall in the backfill did not reduce the distortion. Because the wall was not able to be installed near the quay wall due to existence of gravel and stone filter, the filter material developed a significant magnitude of dynamic earth pressure on the caisson wall, and this pressure in turn pushed the quay wall towards the sea. When the intensity of shaking was increased to 0.43g in centrifuge tests, the expected mitigation was not achieved. When the soft foundation soil was improved, the overlying quay wall was subject to stronger inertial effects. At this moment, therefore, it is concluded that the seismic stability of a gravity-type wall is limited under very strong shaking.

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