SHAKING TABLE TESTS AND EFFECTIVE STRESS ANALYSES ON THE DYNAMIC BEHAVIOR OF WEDGED CAISSONS

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SUMMARY

We have been developing " wedged caisson" as quay wall structures with a high earthquake resistant capacity without an enormous increase in construction cost. The wedged caisson has a bottom that declines towards the ground. Since safety against sliding failure of the caisson increases (to a limit) with the declination angle of the bottom, the width of the caisson can be reduced from that of the traditional caisson. To find the suitable range of the bottom declination angle, a series of shaking table tests using 1/22 scale model caissons with bottom declinations of 0, 5, 10 degrees were carried out. The tests were also simulated numerically using an effective-stress-based fully coupled analysis method, LIQCA, where a cyclic elasto-plastic constitutive model was implemented. From the results of the shaking table tests and the effective-stress-based analysis method, it was found that the seismic capacity of the slender wedged caisson with a bottom declination of 5 degree is comparable to that of the traditional caisson without any bottom declination.

INTRODUCTION

During the 1995 Hyogoken-Nambu Earthquake, most gravity type quay walls suffered tremendous damage due to intense seismic shocks, liquefaction of the replaced soil under the rubble mound of the caisson foundation and liquefaction of the reclaimed land. However, a quay wall designed with a seismic coefficient of 0.25 using the Technical Standard for Port and Harbour Facilities in Japan [Japan Port and Harbour Bureau, 1989] survived the seismic shocks and served as a useful facility for emergency transport. From this experience, the significance of a seismically strong quay wall was widely recognized.

According to the technical standard, the following stability conditions under the seismic shocks have to be evaluated for the design of gravity type quay walls: 1) sliding failure of the quay wall; 2) bearing capacity of the foundation; and 3) overturning of quay wall. Among these conditions, the sliding failure becomes the critical condition for quay walls designed with high seismic coefficient of 0.2 - 0.25. Through trial design of seismically strong quay walls, it was experienced that the width of the caisson becomes large, and ground improvement to support them also becomes costly.

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In late 1995, a simple method to increase safety factor against sliding failure was proposed by the authors, and a joint project was initiated to develop the "wedged caisson" as quay wall structures with high earthquake resistant capacity at low cost. The wedged caisson has a bottom that declines towards the ground. Since safety against sliding failure of the caisson increases (to a limit) with the declination angle of the bottom, the width of the caisson can be reduced from that of the traditional caisson. The purposes of the present study are : 1) to compare the seismic capacity of the wedged caisson with the traditional caisson without bottom declination, and specifically 2) to find the suitable range of the bottom declination angle.

UNDERWATER SHAKING TABLE TEST

Method and Test Conditions

To simulate the behavior of the wedged caisson constructed in sea water, underwater shaking table with a diameter of 5.5 m was used. A table used was implemented at the bottom of a water pool with a 15x15m plane size and a 2 m depth [Sugano et al., 1996]. Three full size quay walls with a design water depth of 12.7 m and bottom declinations of 0, 5 and 10 degrees were designed under the same seismic coefficient of 0.25 following the technical standard [Japan Port and Harbour Bureau, 1989]. As shown in Fig. 1, the resultant caisson widths are decreased with an increase in the bottom declination angles. A similitude for a shaking table test on soil-structure-fluid model in 1G gravitational field [Iai, 1988] was adopted, and a scale factor of 22 (prototype/model) was employed resulting in the similitude summarized in Table 1.

Fig. 2 shows a side view of the test model for Case A3 together with the measurement instrumentation. In each test, three units of model caisson with a 525 mm width in the shore line direction were set together, and the displacement measurement was taken for the central unit, to reduce the effect of the side friction between the back fill and the test container . The model caissons were made of thick aluminum plates and filled with dry sand such that the unit weight of the model caisson becomes 21 kN/m^3. At the bottom of the model caisson, rubber sponge mat was glued to satisfy the frictional factor of 0.7 as assumed in the design of the prototype. The foundation ground, which is assumed to be a firm ground in the prototype, was made from Grade 5 Soma sand mixed with 3% weight jet cement. The rubble mound beneath the caisson was modeled using Grade 4 crushed stones with a range of particle size of 20 -30 mm. After placing 3 model caissons on the crushed stone mound,

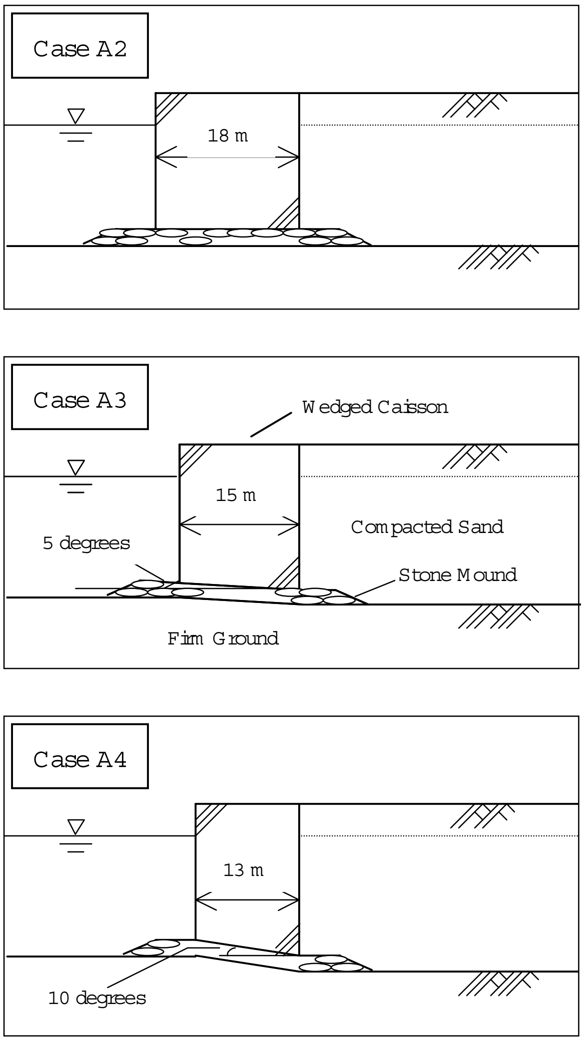


Fig.1: Cross sections of test models.

Table 1: Similitude for shaking table test.

Parameter	Prototype/ Model	Scale
Length	λ	22.0
Density	1	1.00
Time	$\lambda^{0.75}$	10.2
Stress	λ	22.0
Displacement	$\lambda^{1.5}$	103
Velocity	$\lambda^{0.75}$	10.2
Acceleration	1	1.00

accelerometers and porewater pressure meters were installed using thin strings, and subsequently the back was filled and compacted manually with air-dried Grade 5 Soma sand. Then the water level was elevated slowly to a specified water level to saturate the model ground.

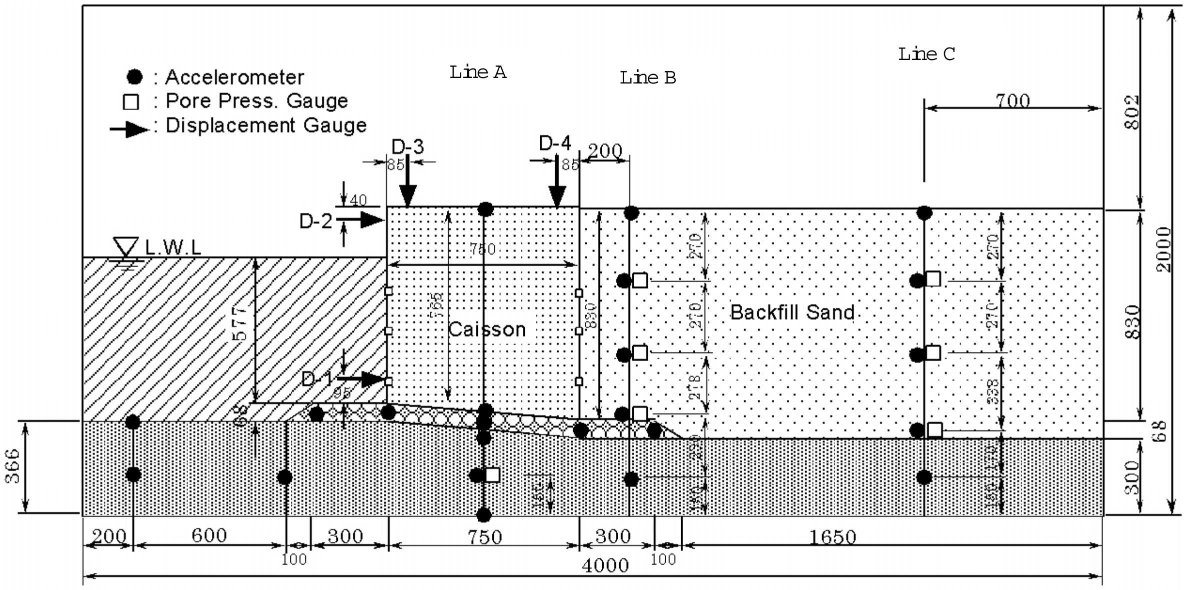
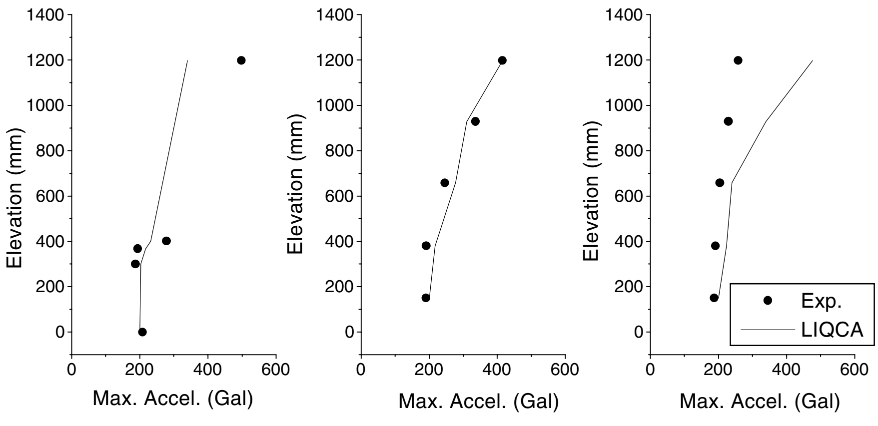
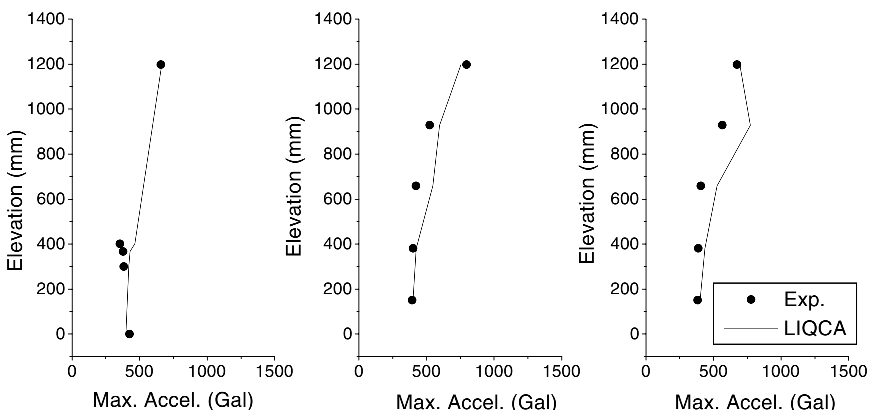


Fig.2: Side view and measurement instrumentation in Case A3



(a) Line A (b) Line B (c) Line C

Fig.3: Measured and calculated maximum acceleration (Case A3: 200 Gal)



(a) Line A (b) Line B (c) Line C

Fig.4: Measured and calculated maximum acceleration (Case A3: 400 Gal)

To compare the seismic capacity of the model caissons, a staged shaking method was adopted where constant amplitude sinusoidal acceleration with a 10 Hz frequency and a 2 s duration (0.98 Hz frequency and 20.32 s duration in prototype) were applied in the direction perpendicular to the shore line. The amplitudes of the acceleration in the first stage was 100 Gal, and then amplitude of 200 and 400 Gal followed. After each excitation, displacement measurements of the model caisson and the ground surface were taken.

Results of Test

Figures 3 and 4 show measured maximum acceleration in Case A3 (bottom declination of 5 degrees) with input accelerations of 200 and 400 Gal, respectively. From the comparison between Fig.3(a) and Fig,4(a), it is observed that maximum acceleration ratio between the top and bottom of the caisson (AH6/AH5) is affected by the input acceleration level, suggesting the seismic isolation effect of the gravity type quay wall under strong excitations.

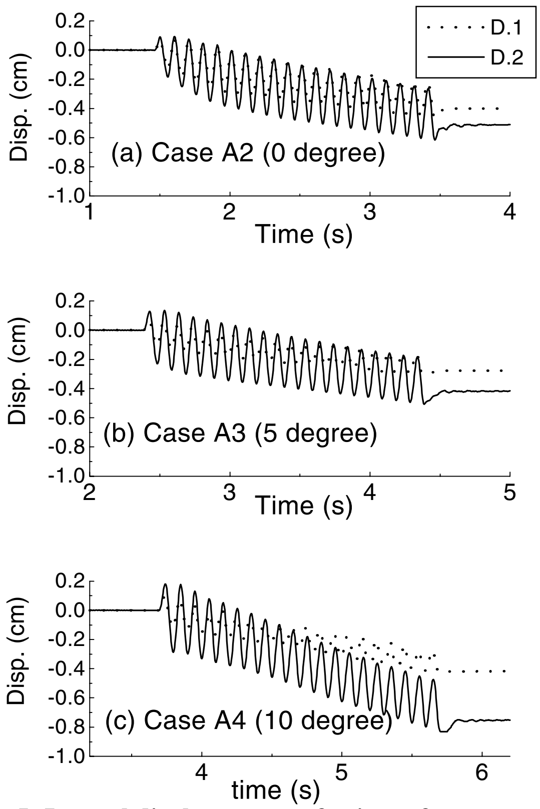


Fig.5: Lateral displacements of caisson from tests (Input = 400 Gal: - sign indicates disp. towards sea)

Figure 5 shows time histories of lateral displacement of caisson from Cases A2, A3 and A4 with input acceleration of 400 Gal. From the figure, it is observed that 1) the caisson is moving gradually towards the sea during the excitation, and that 2) in Case A4, lateral displacement at D-2 (top of the caisson) is developing linearly with time. From the measured displacements such as that shown in Fig.5, residual displacements at the bottom of the caisson and residual rotations of the caissons were calculated and plotted in Figures 7(a) and (b). From these figures, it may be observed that 1) the residual displacements at the bottom of the caisson is not much affected by the bottom declination angle; 2) the residual rotations of the caisson in Case A4 (bottom declination of 10 degrees) is greater than those of Cases A2 and A3; and that 3) there is little difference between the results in Case A2 (0 degrees) and A3 (5 degrees).

EFFECTIVE STRESS ANALYSIS

Method and Conditions of Analysis

Nine cases of shaking table tests (3 models x 3 input motions) as described in Section 2 were simulated numerically using an effective-stress-based fully coupled analysis method, LIQCA, where a cyclic elasto-plastic constitutive model was implemented [Oka et al., 1992; 1994] . Prior to the effective stress analyses, three cases (3 models) of static elasto-plastic analyses considering gravity force were conducted to estimate the initial stresses of the caisson-ground systems.

Table 2 summarizes the material parameters used in the analysis. Among these parameters, are measured parameters unit weights, void ratios, and shear wave velocities. It was found that the plastic modulus parameters of the stone mound greatly affect the calculated residual displacement of the model caisson, from trial calculations on Case A3 with 400 Gal acceleration. Thus these parameters were determined by fitting to the measured residual displacements. The other parameters were estimated based on past experience. It is noted that the overconsolidated ratio, *OCR*, used in the model is a parameter to control the dilatancy of soil, and does not necessarily correspond to the conventional definition of *OCR* [Sekiguchi et al., 1999]. In the analyses, the Rayleigh damping of 0.2% for the stiffness matrix varying with time was assumed. A time step of 0.00025 s (400 step/cycle) was used based on the trial calculations.

Table 2: Material parameters used in the effective stress analysis

	γ(kN/m^3)	k (m/s)	e	λ	κ	Vs (m/s)	M_f	M_m	B_0	B_1	C_f	*OCR*
Stiff Ground[1)]	20.4	0.0	0.55									
Stone Mound	19.4	0.1	0.74	0.03	0.01		1.34	1.16	40000	4000	2000	1.2
Stone Moud/ Caisson [3)]	19.4	0.1	0.74	0.03	0.01		1.16	1.00	40000	4000	2000	1.2
Caisson in Water[2)]	21.0	0.0	0.00									
Caisson in Air[2)]	21.0	0.0	0.00									
Compacted Ssand in Water	19.8	10^{-5}	0.65	0.03	0.01	115	1.51	1.31	2500	250	2000	3.0
Compacted Sand in Air	15.8	10^{-5}	0.65	0.03	0.01	115	1.51	1.31	2500	250	2000	3.0
Compacted Sand in Water/Caisson [3)]	19.8	10^{-5}	0.65	0.03	0.01	115	0.98	0.84	2500	250	2000	3.0
Compacted Sand in Air/Caisson[3)]	15.8	10^{-5}	0.65	0.03	0.01	115	0.98	0.84	2500	250	2000	3.0

Notations:

γ= Unit weght; k = Coefficient of permeability; λ = Compression index; κ = Swelling index;
Vs = Shear wave velocity; M_f = Stress ratio parameter corresponds to failure angle;
M_m = Stress ratio parameter corresponds to phase transformation angle;
B_0, B_1, C_f = Plastic modulus parameters; *OCR* = Overconsolidatio ratio

[1)] Elastic material was assumed (Young's modulus = 500 MPa; Poisson's ratio = 0.45)

[2)] Elastic material was assumed (Young's modulus = 7200 MPa; Poisson's ratio = 0.34)

[3)] Thin one layer elements were used as interface between the model caisson and the ground materials

Results of Analysis

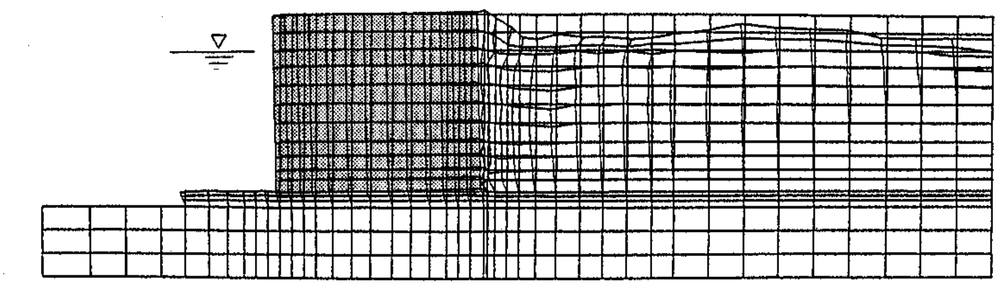
Figures 3 and 4 compare the measured and calculated maximum accelerations in Case A3 where the bottom declination angle is 5 degrees. It is observed that the agreement is good at the measurement line B in Fig. 3 (200 Gal); and at all the measurement lines in Fig.4 (400 Gal). In Cases A2 and A4, the same tendency was recognized.

Figure 6 shows calculated residual displacement in three cases after the excitation of 400 Gal. From these figures, it is observed that the caisson is moving transiently towards the sea with an anti-clockwise rotation as was observed in the shaking table tests. The relationships between shear stress and strain in the stone mound shown in Fig.6(b) clearly indicate that the movement of the caisson is induced by the accumulated residual shear strain in the stone mound. It is noted that the deformation of the upper portion of the fill (two FE layers above the water table) is overestimated, because the portion is modeled as a completely dry non-linear material.

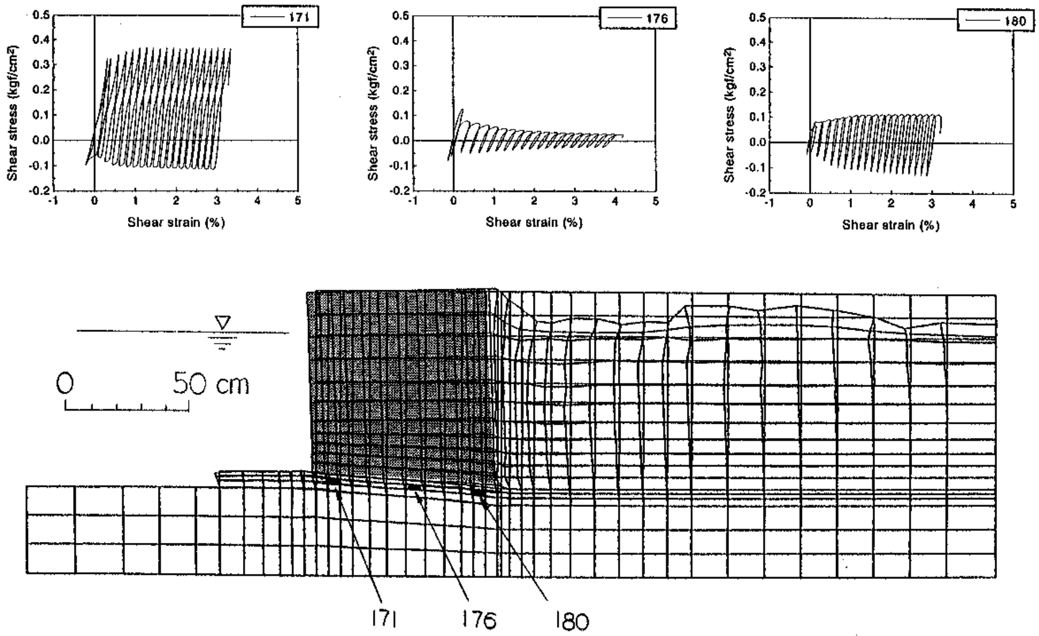
Figures 7(a) and (b) compare the calculated and measured residual displacements at the bottom of the caisson and residual rotations of the caissons. From these figures, it may be observed that 1) the calculated results are consistent with those measured in the shaking table test; 2) the seismic resistant capacity of the slender wedged caisson with bottom declination angle of 5 degrees is practically identical to that of the traditional caisson with a flat bottom.

CONCLUSIONS

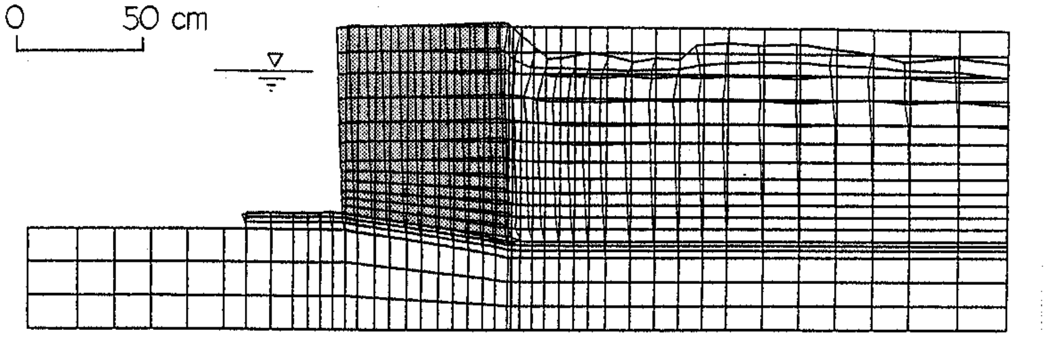
We have been developing " wedged caisson" as economical quay wall structures with high earthquake resistant capacity. To study the suitable range of the bottom declination angle, a series of underwater shaking table test using 1/22 scale model caisson with bottom declinations of 0, 5 and 10 degrees were carried out. From the shaking table tests, it was found that: 1) the residual displacements of the model caisson with a 5 degree bottom declination were comparable to those without bottom declination; and that 2) the displacements of the model caisson with a 10 degree bottom declination were greater than those of the other two models.



(a) Case A2 (bottom declination angle is 0 degrees)

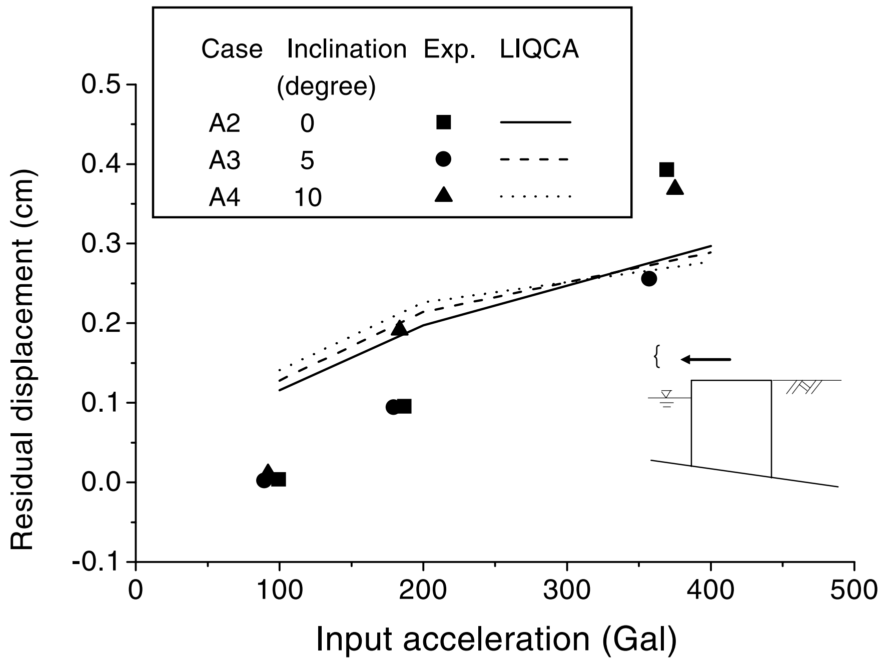


(b) Case A3 (bottom declination angle is 5 degrees)

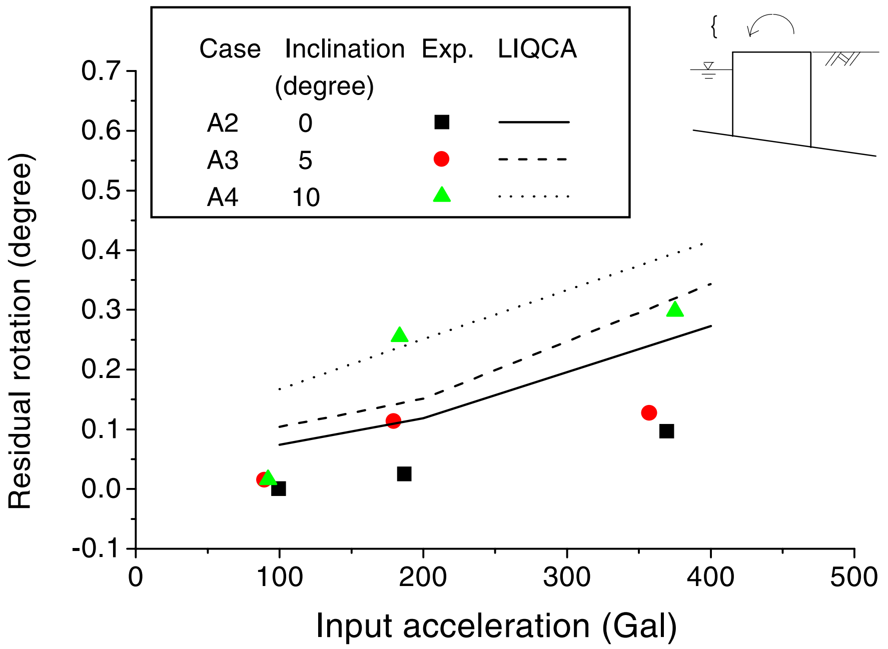


(c) Case A4 (bottom declination angle is 10 degrees)

Fig.6: Calculated residual deformations of caisson-ground systems. (400 Gal; Displacement scale is 5 times as much as geometric scale)



(a) Residual displacement at the bottom of the caisson



(b) Residual rotation of the caisson

Fig.7: Comparison of measured and calculated residual movements of model caissons.

The shaking table tests were simulated numerically using an effective-stress-based fully coupled analysis method, LIQCA, where a cyclic elasto-plastic constitutive model was implemented . From the results of the analyses it was found that: 1) the calculated acceleration and residual displacements agreed well with those observed with the suitable choice of soil parameters; and that 2) the seismic capacity of a caisson with a 5 degree bottom declination was comparable to those without any declination, as was suggested from the results of shaking table tests.

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