EXPERIMENTAL STUDY OF THE BEHAVIOR OF HYBRID (STEEL-CONCRETE COMPOSITE) CAISSON-TYPE QUAY WALLS DURING EARTHQUAKES USING AN UNDERWATER SHAKING TABLE

Yoshio SHIOZAKI[1], Takahiro SUGANO[2], Shunsuke YAMAMOTO[3], Tsuyoshi TANAKA[4] And Koji SEKIGUCHI[5]

SUMMARY

The Hybrid Caisson is composed of outside-wall slabs, bottom slabs and footing slabs, all made of a steel-concrete composite, and bulkheads made of stiffened steel plates. By using steel members, the footing can be extended, which is effective for special conditions such as deep water, poor subsoil, and others. In designing quay walls or sea walls, current design standards on port and harbour structures in Japan assumes that the backfill rubble above the landside footing is a part of gravity structure resisting against active earth pressure. Therefore, by extending the footing, the caisson width can be reduced. To study the behavior of backfill rubble above the landside footing, and to ascertain the seismic stability of the structure, we made a test using an underwater shaking table. The test was also simulated numerically using an effective stress analysis method, FLIP. From the present study, it is concluded that the Hybrid Caisson-type quay wall designed using current standards with a seismic coefficient of 0.25 has sufficient seismic stability for the level of design earthquake

INTRODUCTION

Many caisson-type quay walls in Kobe were completely destroyed by the 1995 Hyogoken-Nambu Earthquake. Therefore a seismic performance verification-type design method was introduced in the Technical Standard for Port and Harbour Facilities and Commentaries revised in 1999 [Bureau of Ports and Harbours, Ministry of Transport, Edit., 1999]. The earthquake-resistant verification becomes an important issue for the Hybrid Caisson-type quay walls of which airscape is shown in Fig.1. The Hybrid Caisson is composed of outside-wall slabs, bottom slabs and footing slabs, all made of a steel-concrete composite, and bulkheads made of stiffened steel plates. By using steel members, the footing can be extended, which is effective for deep water and poor subsoil conditions. When quay walls or sea walls are designed using the Technical Standard, it is assumed that the backfill rubble above the landside footing is a part of gravity structure resisting against active earth pressure and inertia force as shown in Fig.2. In the figure, the shaded part is assumed as a gravity structure. The input earthquake motion for seismic performance verification is comparable to those obtained during the Hyogoken Nambu Earthquake when the high seismic resistant quay wall is designed with a seismic coefficient of 0.25. However the validity of the assumption shown in Fig.2 has not been proven for the strong motions. To study the behavior of backfill rubble above the landside footing, and to ascertain the seismic stability of the structure, we made a test using an underwater shaking table. The test was also simulated numerically using an effective stress analysis method, FLIP

[1] *Applied Technology Research Center, NKK Corporation, Kawasaki, Japan email : yshiozak@lab.keihin.nkk.co.jp*
[2] *Port and Harbour Research Institute, Ministry of Transport, Yokosuka, Japan*
[3] *Japan Port Consultants Ltd., Tokyo, Japan*
[4] *Japan Science and Technology Corporation, Kawaguchi, Japan*
[5] *Applied Technology Research Center, NKK Corporation, Kawasaki, Japan email : yshiozak@lab.keihin.nkk.co.jp*

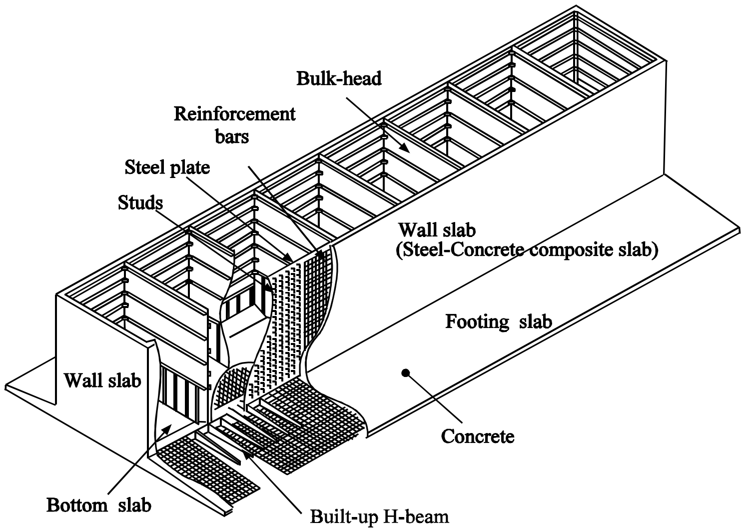


Fig.1: The airscape of the Hybrid Caisson

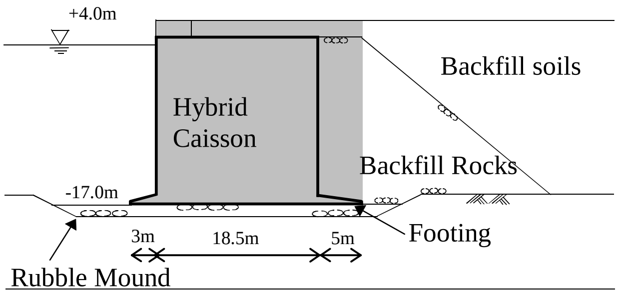


Fig.2: The prototype of the Hybrid Caisson-type quay wall

UNDERWATER SHAKING TABLE TEST

Method and Test Condition

The prototype of the Hybrid Caisson-type quay wall taken up by the experiment is shown in Fig.2, whose size is 21m in height, 18.5m in width, with a 5m width landside footing and 3m width seaside footing. In the test, the caisson-type quay wall was modeled at a scale of 1/25 (λ=25) of the prototype. The quay wall model including foundation and backfill soils was made in a 3.5m long, 1.5m wide and 1.5m deep steel container on a shaking table, which was set in the middle of a water pool with a 13m by 13m plane size and a 2m depth to simulate the effect of sea water.

The experiment techniques adopted here were the same as those developed by Sugano [et al, 1995] to analyze the mechanism of the collapse of the caisson-type quay walls damaged by the Hyogoken-Nambu Earthquake. We adopted a similitude for shaking table test on soil-structure-fluid model in 1G gravitational field [Iai, S., 1988], and the scaling factors for this test are shown in Table 1.

Table1: Scaling factors for shaking table test

Quantities	Scaling factors	Scaling factors for 1/25 model
Length	λ	25.00
Density	1	1.00
Time	$\lambda^{0.75}$	11.18
Acceleration	1	1.00
Stress	λ	25.00
Displacement	$\lambda^{1.5}$	125.00
Pore Water Pressure	λ	25.00

The cross-section of the test model is shown in Fig.3 together with the measurement instrumentation. In the test, three units of model caisson with 480mm width in the shore line direction were set together, and the displacements and accelerations were measured with the central unit, to reduce the effect of the side friction between the back fill and the test container. The model caisson was made of thick aluminum plates and filled with dry sand and sinker so that the dry density of the model caisson became 21KN/m^3. Fig.4 shows the model caisson for the measurement, which is able to measure total earth pressures through load cells. At the bottom of the model caissons, rubber sponge mat was glued to satisfy the friction 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 was modeled using Grade 4 crushed stones. The backfill rubble was modeled using Grade 6 crushed stones. The backfill soil was made from Grade 5 Soma sand and compacted sufficiently not to liquefy. In order to measure the dynamic performance of the quay wall during the shaking, accelerometers, pore pressure transducers, and displacement transducers were installed.

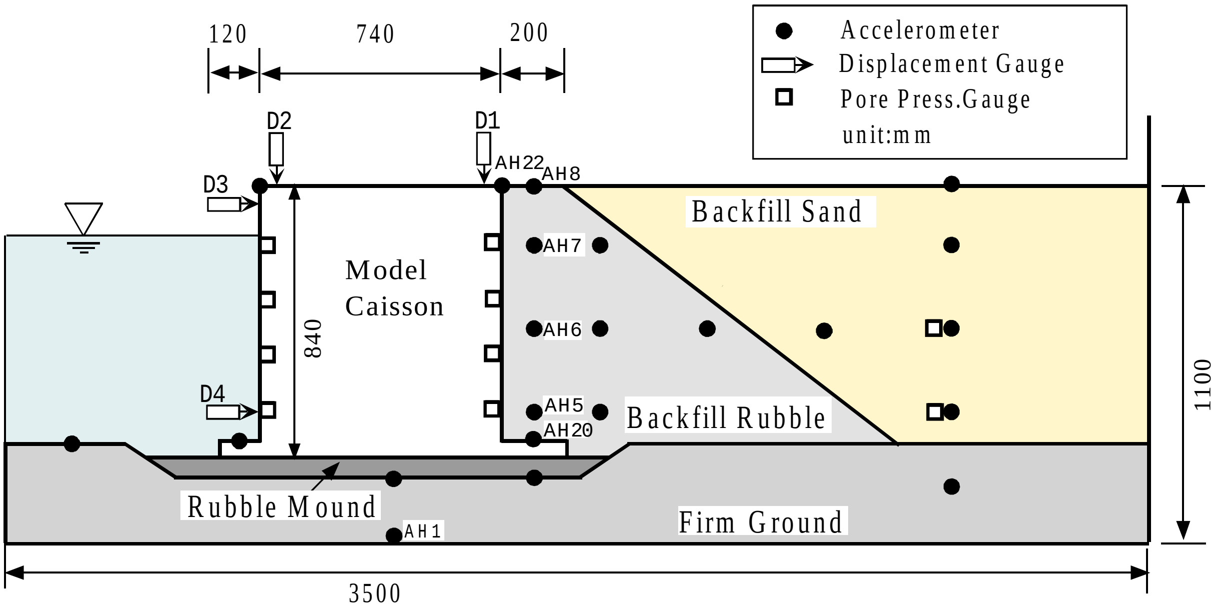


Fig.3: Cross-section of the test model

The input motion used in the test was a strong motion record from the Hyogoken-Nambu earthquake collected at GL.-32m in the Port Island. The peak accelerations were 5.44, 4.61, and 2.00 m/s^2 in NS, EW and UD directions, respectively. The input motion was applied in accordance with the direction of the quay wall facing west.

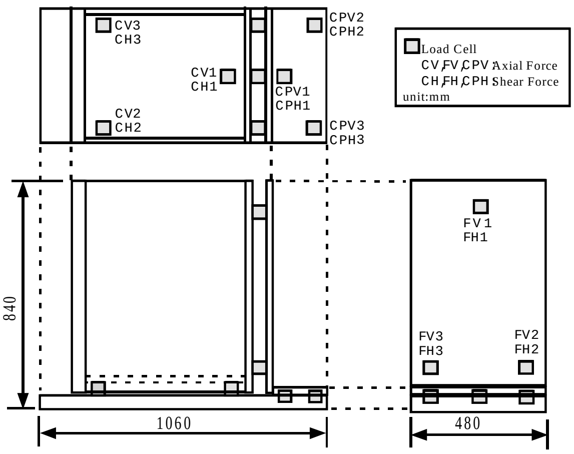


Fig.4: Model caisson for the measurement

Results of Test

Residual displacement of model caisson-type quay wall is shown in Fig.5, scaled in terms of prototype. The horizontal residual displacement of the caisson top (D3) was 11.8cm to the seaside and vertical residual displacement (D2) was 6.5cm downward. These displacements are much smaller than those of the caisson-type quay walls damaged during the Hyogoken-Nambu Earthquake; and it is possible to judge that the Hybrid Caisson-type quay wall has sufficient earthquake-resistant capacity.

Fig.6 shows time histories of the response of the model. From the figure, we can see that the acceleration (AH20, AH5- AH8, AH22) of backfill rubble on the landside footing is behaving by the same phase, although they are amplified with the elevation. It becomes clearer when piling up the response displacement histories obtained by integrating these accelerations (AH20, AH5- AH8, AH22) twice as shown in Fig.7. Judging from these results, it is confirmed that the model caisson and the backfill rubble above the footing moved together under the strong motion for a seismic performance verification-type design.

The vertical load CPV (total value of CPV1-3) which acted on the surface of the landside footing was changing during the earthquake shaking, but on the average, the value of the load before the shaking continued to act. Fig.8 shows the distribution (the dynamic ingredient) of the load cell force at the time 1.814sec when the horizontal load FV (total value of FV1-3) took maximum. As shown in the figure, the model caisson moved to the seaward as tilting forward, with the increase in the vertical load at the seaside (total value of CV2 and CV3). It is also found that the horizontal load FV which acted on the back of the caisson was increasing, and that the vertical load CPV was increasing too (see also Fig.6). From above results, we can conclude that the backfill rubble above the footing was working as the effective weight.

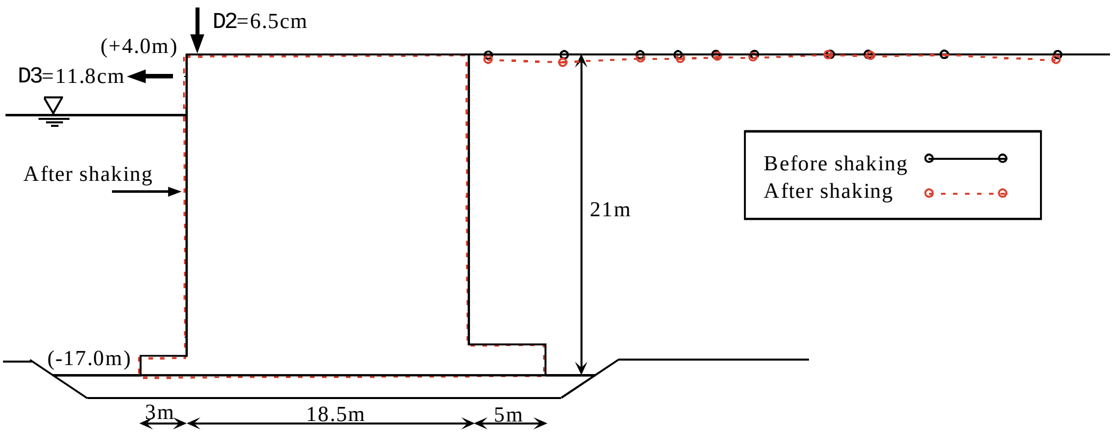
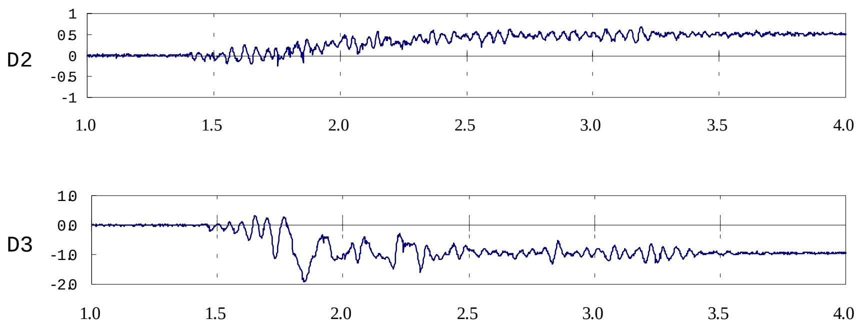
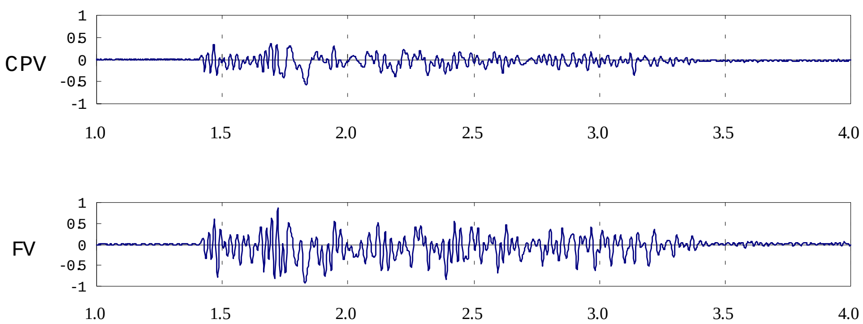


Fig.5: Residual displacement of model caisson-type quay wall

Disp [mm] (D2:+sign indicates disp. towards down; D3:-sign indicates disp. towards sea)



Force [KN] (CPV,FV:-sign indicates compression force)



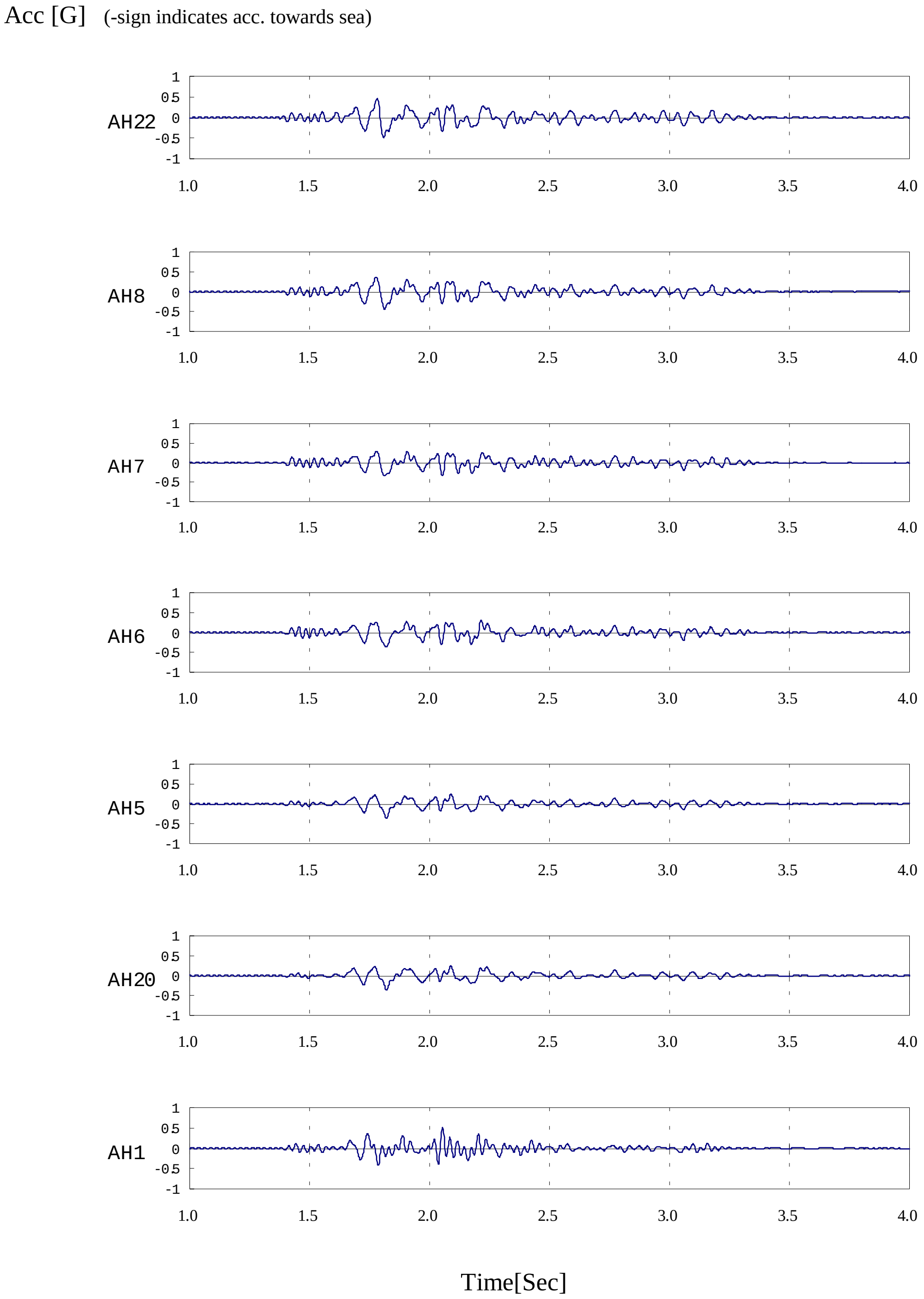


Fig.6: Time histories of the response of the model

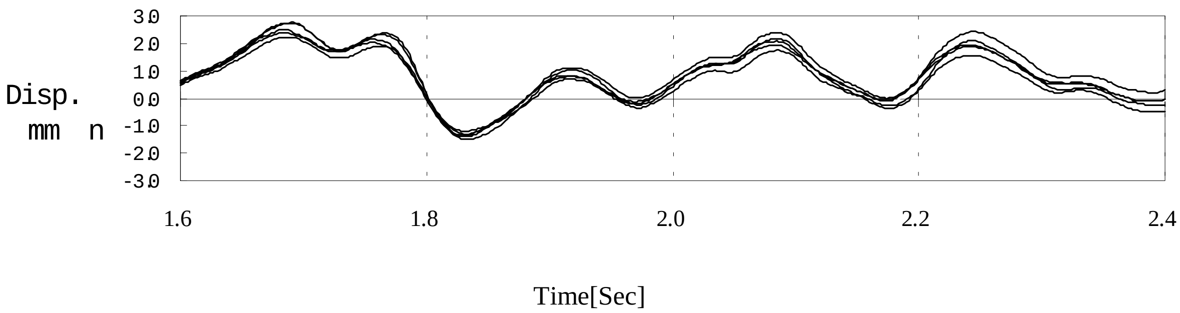


Fig.7: Displacement time histories obtained by integrating accelerations (AH20, AH5- AH8, AH22)

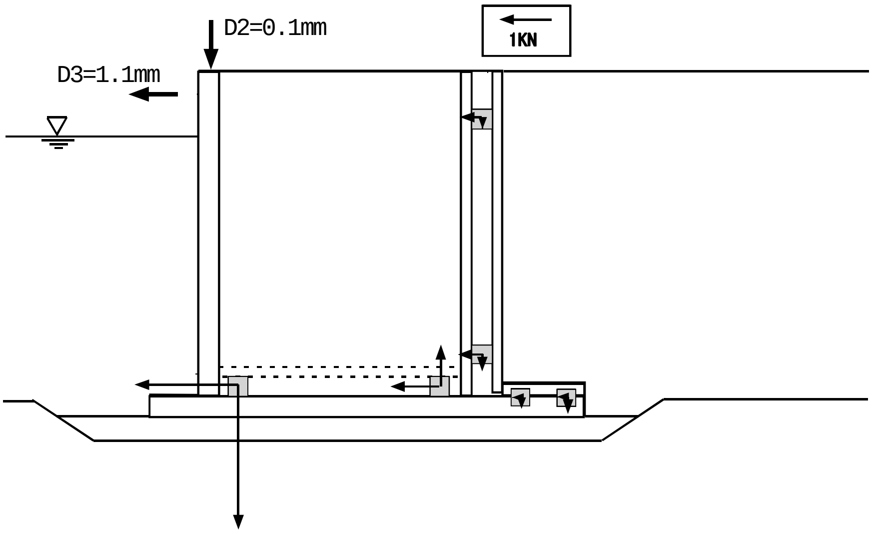


Fig.8: Dynamic ingredient of the load cell force at the time of 1.814sec

EFFECTIVE STRESS ANALYSIS

Method and Conditions of Analysis

Shaking table test was simulated numerically using an effective stress analysis method, FLIP [Iai, S, et al, 1990]. The material parameters used in the analysis are shown in Table 2, which were decided based on densities, void ratio, shear wave velocity and experience in the former analysis [Iai, S., et al, 1995]. Backfill rubble, rubble mound and backfill soils were modeled with multi-spring elements [Iai, S., et al, 1990]. The model caisson and foundation ground were modeled with plane linear elements. Joint elements were installed between the caisson and soils. The input motions used in the analysis were the horizontal and vertical accelerations recorded at the shaking table in the model test described in section 2.

Table 2: Material parameters used in the analysis

	ρ(KN/m^3)	σ_{m0}'(kPa)	G_{m0}(kPa)	K_{m0} (kPa)	φ_f'(deg.)
Backfill sand in Air	15.5	0.82	21119	55076	38
Backfill sand in Water	19.3	3.67	20541	53567	38
Backfill rubble in Air	15.9	0.84	26323	68649	40
Backfill rubble in Water	19.6	3.45	32526	86837	40
Foundation rubble	18.3	98.0	4900	12779	40

ρ: Density; σ_{m0}': Initial confining stress; G_{m0}: Initial shear modulus; K_{m0} : Initial bulk modulus; φ_f': Friction angle

Results of Analysis

Fig.9 shows calculated residual deformation. The vertical displacement at the seaside of the caisson becomes small compared with the experimental value; however the seaward residual displacement of the caisson generally agrees with the quantities obtained from the experiment. Fig.10 shows calculated time histories of displacement and acceleration together with the experimental values. The calculated time histories agree well with the quantities obtained from the experiment. From these comparisons, we can say that the shaking table test could be simulated by using FLIP with appropriate setting of the material parameters.

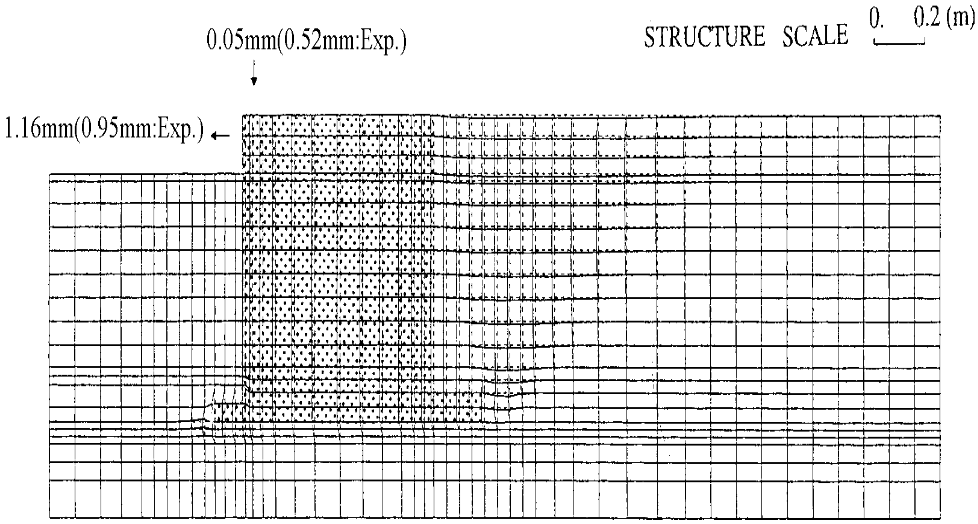


Fig.9: Calculated residual deformation

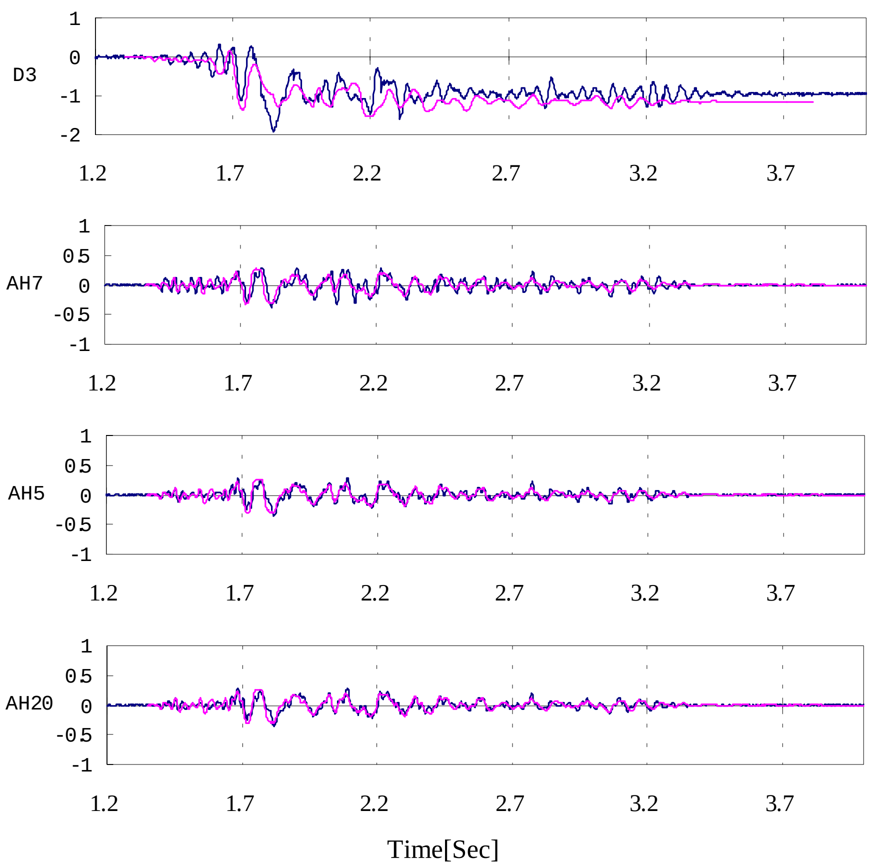


Fig.10: Comparison of measured and calculated time histories

CONCLUSIONS

To study the behavior of backfill rubble above the landside footing, and to ascertain the seismic stability of the Hybrid Caisson-type quay wall designed with a seismic coefficient of 0.25, we made a test using an underwater shaking table test. The test was also simulated numerically using an effective stress analysis method, FLIP. From the analysis of the test results and the numerical simulation, the following conclusions are drawn:

1)The residual displacement of the model caisson was small after the excitation using the strong motion record from the Hyogoken-Nambu Earthquake; and thus, it is concluded that the Hybrid Caisson-type quay wall designed using the current Standard has sufficient seismic stability for the level of design earthquake.

2)The model caisson and the backfill rubble above the landside footing moved together; and backfill rubble above the landside footing was working as the effective weight.

3)We found that the calculated displacements and accelerations agree well with those measured by using FLIP with appropriate setting of the material parameters.

The research work presented in this paper was conducted as part of “Joint Research Project on the Seismic Stability of Gravity-type Quay Walls”, between Port and Harbour Research Institute, Ministry of Transport and NKK Corporation.

REFERENCES

Bureau of Ports and Harbours, Ministry of Transport, Edit.(1999), *Technical Standards for Port and Harbour Facilities and Commentaries*.(In Japanese)

Iai, S.(1988), Similitude for shaking table tests on soil-structure-fluid model in 1G gravitational field, *Report of the Port and Harbour Research Institute*, Vol.27, No.3, pp.3-24 .

Iai, S, Matsunaga.Y. and Kameoka.T.(1990), Parameter identification for a cyclic mobility model, *Report of the Port and Harbour Research Institute*, Vol.29, No.4, pp.57-83.

Iai, S., Ichii, K. and Morita, T.(1995), Mechanism of damaged to port facilities during 1995 Hyogo-ken Nanbu Earthquake.(Part 7: Effective stress analysis on a caisson type quay wall), *Technical Note of the Port and Harbour Research Institute*, No.813, pp.253-279. (In Japanese)

Sugano, T., Mitoh, M. and Oikawa, K.(1995), Mechanism of damage to port facilities during 1995 Hyogo-ken Nanbu Earthquake.(Part 6: Experimental study on the behavior of caisson-type quay wall during earthquake using underwater shaking table), *Technical Note of the Port and Harbour Research Institute*, No.813, pp.207-252. (In Japanese)