Seismic Performance Evaluation of Quay Walls by Shaking Table Tests

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INTRODUCTION

The shaking table tests for 5 different model sections are performed to investigate the behaviors of quay walls during earthquakes and to evaluate the seismic performance of quay walls with countermeasures. Five different model sections are designed varying the foundation soil densities and the backfill materials. The relative densities of foundation soils are varied to be 20% and 90%, and gravels as well as light weight aggregates are used as countermeasure backfill materials. Sand compaction piles are also installed in the backfill as another countermeasure. Pore water pressures and the magnitude of accelerations and deformations are measured in quay walls and grounds for dynamic analysis.

SHAKING TABLE TEST PROGRAM

A test box is made of plexi-glass, whose size is $192^{cm} \times 44^{cm} \times 60^{cm}$(height). Fig. 1 shows 5 different model sections. In Case 1 and Case 2, dense foundation and loose foundation are prepared respectively without countermeasures. In Case 3, gravel backfill is prepared behind quay wall. In Case 4 and Case 5, light weight backfill and sand compaction piles are applied respectively for seismic countermeasures.

12 piezometers, 4 accelerometers, and 2 LVDT's are instrumented as shown in Case 1-3 of Fig. 1. The average particle size of model sands(D_{50}) is 0.17mm, and the coefficient of uniformity(c_u) is 1.9. The uniform gravels, whose size varies between 3mm and 5mm, are used for gravel backfill and mound. The saturated unit weight of light material is 1.25 t/m^3 and it has the uniform particle size of about 1cm.

Loose sand ground is formed in the water by pouring the sand through 1mm sieve to control the falling velocity and to make it loose as possible. The relative density of loose sand ground(D_r) is about 20%. A dense sand layer, whose relative density is about 90%, is prepared by shaking. The sand compaction piles are installed using pipe casings, a vibrating hammer, and compaction rod. Model quay wall is made of wood and the distance between the side walls of quay wall and test box is maintained by 2mm to prevent the frictional resistance.

Models were shaken for 5 seconds with 4Hz frequency. The amplitude of input acceleration is increased by 0.05g from 0.10g.

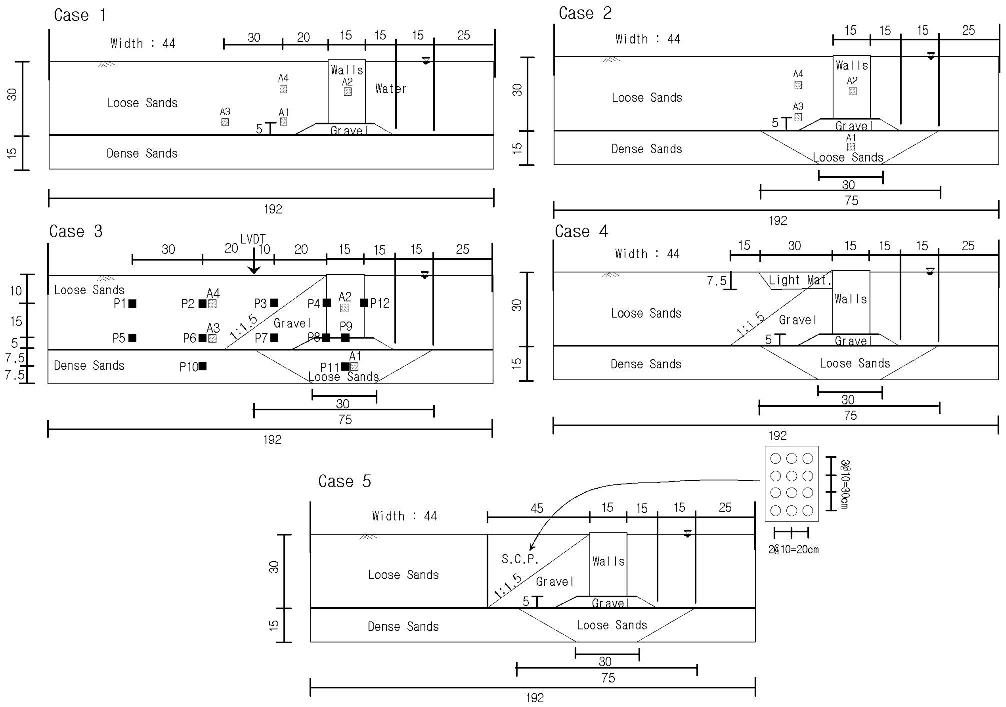

Fig. 1 Test model section and instrumentation

TEST RESULTS

Seismic Behavior of Quay Walls

Fig. 2 shows the response of excess pore pressure and accelerations in Case 1. Piezometer P3 and accelerometer A4 are installed at the same place in backfill soils, and accelerometer A2 is installed at the center of gravity of quay wall. The liquefaction of backfill soils occurred after about 2.0 sec, then the acceleration of quay wall was biased toward (+) value. Therefore, it is concluded that quay wall failed due to the increase of soil pressure caused by the softening of backfill soils.

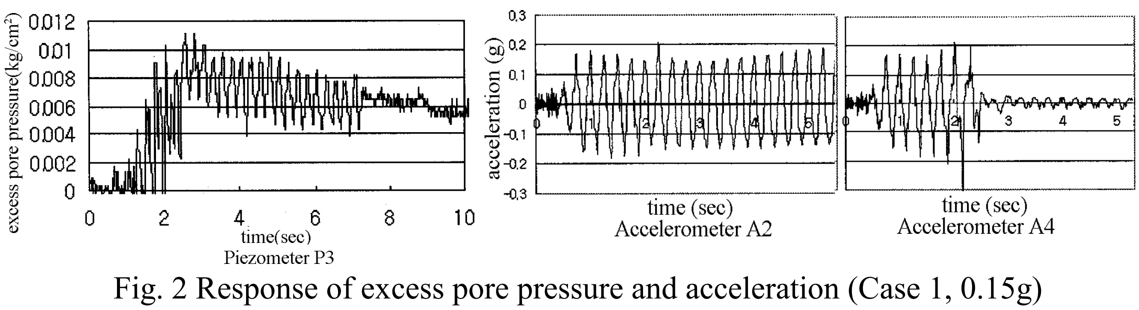

Fig. 2 Response of excess pore pressure and acceleration (Case 1, 0.15g)

When the amplitude of input acceleration is 0.10g, no quay walls failed for all 5 cases. In Case 1(dense foundation) and Case 2(loose foundation), the sliding failure of quay walls occurred at 0.15g. In Case 3(gravel backfill), sliding failure occurred with a little rotation of quay wall at 0.15g. In Case 4(light weight backfill) and Case 5(sand compaction piles), the failure of quay wall occurred at 0.20g. In Case 4 rotation failure occurred and in Case 5 sliding failure occurred.

Table 1 shows the settlement of backfill soils during and after shaking. Most of settlement occurred during shaking and the additional settlement, about 12.7% of total settlements occurred during the dissipation period of excess pore pressure after shaking is stopped.

Fig. 3 shows the relationships between the settlement of backfill soils and the magnitude of the lateral displacement of quay wall, which are found to be almost linear. In case of sliding failure(Case 1, Case 2, Case 3 and Case 5), the settlement of backfill and the magnitude of the lateral displacement of quay wall occurred as the ratio of 1 to 2.8(R^2=77%) and in case of rotation failure(Case 4) as the ratio of 1 to 1.3(R^2=90%). In Case 4 and Case 5, the deformation of quay wall as well as backfill soils is significantly reduced at the same level of input acceleration comparing with the cases without any countermeasure. Therefore, light weight backfill and sand compaction piles are thought to be effective to improve the seismic performance of quay walls.

Table 1. Settlement of backfill (mm)

		Settlement of Backfill	
		During Shaking	After Shaking
Case 1	0.15g	20.4	3.1
Case 2	0.15g	24.4	1.8
	0.20g	15.7	2.1
Case 3	0.15g	17.1	1.6
	0.20g	15.0	2.7
Case 4	0.15g	0	2.1
	0.20g	25.3	2.1
	0.25g	13.8	2.1
	0.15g	1.2	0.2
	0.20g	8.5	0.6
Case 5	0.25g	12.6	1.2

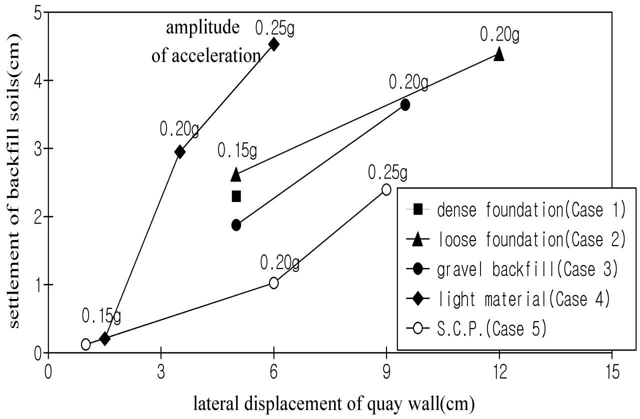

Fig. 3 Relationship between settlement of backfill soils and lateral displacement of quay wall

Response of Pore Water Pressure

Fig. 4 shows the contours of excess pore pressure ratio when the excess pore pressure in backfill soils reaches the maximum.

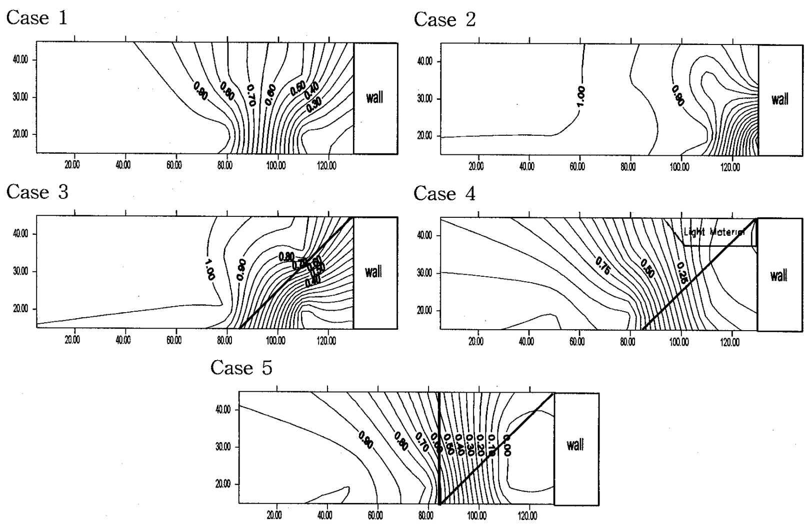

Fig. 4 Contours of excess pore pressure ratio in backfill soils at 0.15g

The liquefaction of backfill soils occurred at 0.15g for all cases and the farther from the wall, the higher excess pore pressure developed due to the reduction of confining pressure near wall caused by the outward movement of quay wall during shaking. In Case 3, quay wall failed at 0.15g despite of the presence of gravel backfill because the contours of excess pore pressure in Case 3 are similar with that in Case 2. In Case 4 and Case 5, no quay walls failed at 0.15g. In Case 4 the reason is thought to be that the large grain size of light material caused an drainage effects and light material reduced the total soil pressure acting on the wall, and in Case 5, that soil stiffness was increased by compaction and excess pore pressure was reduced in backfill soils.

Response of Hydrodynamic Pressure

The hydrodynamic pressure(p_w) is generally estimated by Westergaard approximate solution(1931). Fig. 5 shows good agreement between observed and evaluated values, which confirms the reliability of Westergaard solution.

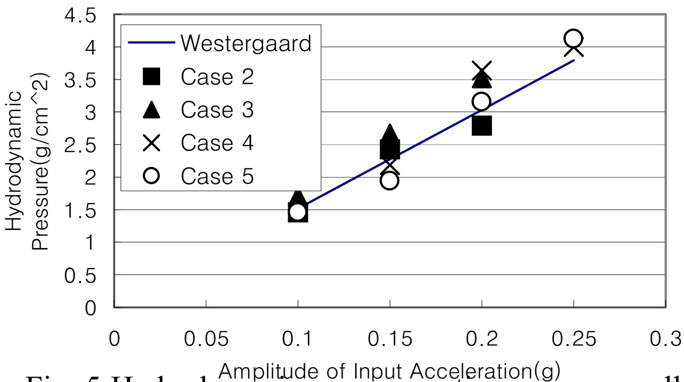

Fig. 5 Hydrodynamic pressure action on quay wall according to the amplitude of input acceleration

CONCLUSIONS

1) The failure of quay walls under the earthquake loading occurs due to the increase of total soil pressure caused by the softening of backfill soils.
2) The farther from the quay wall, the higher excess pore pressure developed for all cases, whose reason is thought to be that confining pressure near the wall is reduced by the outward movement of quay wall during shaking.
3) The additional settlement has occurred during the dissipation period of the excess pore pressure after the shaking is stopped, and the settlement of backfill is found to be approximately proportional to the magnitude of the lateral displacement of quay wall.
4) In cases where light weight backfill and sand compaction piles are applied, the deformation of quay wall as well as the backfill soils is significantly reduced at the same level of input acceleration comparing with the cases without any countermeasure. Therefore, it is concluded that those countermeasures are effective in improving the seismic performance of quay walls.
5) The hydrodynamic pressures estimated by Westergaard solution show good agreement with the observed values, which confirms the reliability of Westergaard solution.

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