Shaking Table Test of Waterfront Structure for Liquefaction Countermeasure

항만구조물의 액상화 대책을 위한 진동대 실험에 대한 연구

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Abstract

Liquefaction leads to severe damage to earth structures after an earthquake. In this study, shaking table tests were performed on model waterfront structures as a countermeasure against liquefaction. The waterfront structure was reinforced by a compacted zone, which was investigated for its effectiveness in protecting the structure from excessive deformation induced by the lateral pressure of liquefied ground. Through the tests on embankment, double sheet pile wall, and anchor sheet pile wall, good quantitative information on the behavior of flow failure and the extent of reinforcement was obtained. The extent of a compacted zone for the protection of the structure depends on the magnitude of the acceleration during the shaking. The measured deformation was represented in terms of the extent of the compacted zone and the magnitude of the input acceleration.

요 지

지진으로 인한 액상화현상은 토목구조물에 막대한 피해를 주고 있다. 본 연구에서는 액상화현상에 대한 대책을 연구할 목적으로 모형 항만구조물을 대상으로 하여 진동대 실험을 실시하였다. 액상화현상으로 항만구조물에 발생하는 과잉변형을 방지하기 위하여 보강구간을 설치하고 그 효과에 대하여 검토하였다. 제방, 이중 널 말뚝벽과 앵커 구조물의 진동대 실험을 통하여, 액상화 지반의 유동변형에 대한 특성과 보강범위에 대한 정량적 자료를 얻었다. 항만구조물을 보호하기 위한 보강구간의 범위는 진동가속도의 크기에 따라 다르다. 실험을 통하여 얻어진 구조물의 과잉변형을 보강구간의 범위와 진동가속도의 크기에 따라 나타내었다.

1. Introduction

Earthquakes have induced a large amount of damage on civil engineering facilities, such as the settlements of buildings, landslides and the failures of earthdams. It has been known that soil liquefaction is the major cause leading to these kinds of damage. On liquefied

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ground, permanent deformations were reported to spread over a wide area which sometimes ranges from a few meters to over a hundred meter. Specially, it is noticed that a large lateral flow can easily occur at some ground conditions such as (a) flat ground in front of a surcharge such as dykes and embankments, (b) flat ground behind a waterfront such as rivers, lakes and sea, and (c) slightly sloped ground[1~4].

Methods of countermeasure for reducing damage from liquefaction can be classified into two groups[6].. The first method is to make some reinforcements on the loose ground deposit not to occur the liquefaction during earthquakes. The second is to allow the liquefaction, but the structure is designed strongly enough to resist the liquefied ground. A different method is applied to a different type of structure. For example, waterfront structures are constructed on ground reinforced by densification, but on the other hand, buried pipelines are generally installed in the natural deposit without ground treatments.

Recently, shaking table tests have been widely performed to investigate the behavior of ground during earthquakes. Particularly, the model grounds with slopes are observed with emphasis on the behavior of the flow failure induced by liquefaction[3, 7, 8]. In this study, shaking table tests were performed on model waterfront structures as a countermeasure against liquefaction. The waterfront structures were reinforced by compacted zones. The effectiveness of a compacted zone is examined for protecting the structure from excessive deformation caused by liquefaction. Determination of the optimum area of the reinforcement is the major interest in the design of structures built on liquefiable ground.

2. Flow failure of liquefied ground

Liquefaction induces two types of failure, flow failure and deformation failure on inclined ground. Flow failure is defined on the condition where a soil mass can deform continuously until equilibrium is restored only after enormous displacement and settlement. Generally, it occurs near the end of seismic shaking or in quite a period immediately after shaking. Hence, deformation failure involves unacceptably large permanent displacement or settlement during shaking, but the ground remains in a stable condition following shaking without great change in geometry.

The failures by the liquefaction occur to the ground having driving shear stresses due to the existence of embankment and the foundation of structure[2, 5, 9]. Ishihara et al.[2] have described the stability check and the extent of deformation as follows. Consider a sloping ground subjected to the initial shear stress τ_s. The schematic undrained stress−strain behavior of the ground can be illustrated by Fig. 1, where S_u is defined as the residual shear strength and $\tau_{d\ell}$ is the peak shear strength which is commenly defined under the stress conditions of 5 cycles. Suppose an earthquake hits a ground inducing a seismic shear stress τ_d within a soil mass. If the induced shear stress is larger than the peak shear strength $\tau_{d\ell}$, liquefaction is triggered. Then, a progressive failure starts and the shear resistance keeps on decreasing to the steady−state or residual strength. Furthermore, when the undrained steady−state strength required for static equilibrium is smaller than the shear stress, a large flow slide is generated.

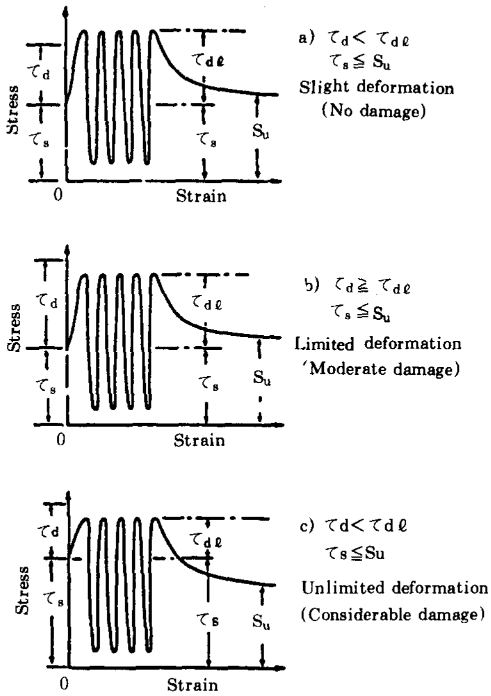


Fig. 1 The stability of liquefied ground.

Recently, shaking table tests have been widely performed to investigate permanent deformation induced by liquefaction because of the advantages that ground deformation can be observed directly during testing. Based on the shaking table tests on sloped ground, Towhata et al.[8] pointed out several important features regarding the permanent deformation of liquefied sand. First, the gradient of the liquefied sand layer influences directly the permanent ground deformation. Second, when a ground surface is level, permanent lateral displacement is small even if the bottom of the liquefied ground is inclined. Third, the liquefied sand behaves like either a liquid or a very soft solid material. Thus, permanent displacement continues until the slope becomes a level ground, or when a force equilibrium condition is reached at its inclined configuration.

Ishihara and Takeuchi[3] performed large size shaking table tests on model dykes laid on liquefiable sand deposits. Three tests were carried out on the clay dykes, which were placed on the loose sand deposit having a compacted zone. Installation of the compacted zone was effective only when it was built in front of a liquefiable zone covered with a sufficiently thick surcharge. Based upon the observations from the shaking table test and the actual failure of the tailing dam, the existence of a slip surface was assumed within the liquefied layer. Along the slip surface, the sliding of the tailing dam was considered to take place.

3. Shaking table unit and the model structure

3. 1. Test equipment

The shaking table at the soil laboratory of the University of Tokyo consists of several units. The system of the equipment is shown in Fig. 2, and it contains the following units: an input unit for cyclic waves, control unit, oil pressure unit, actuator, shaking table, and soil tank.

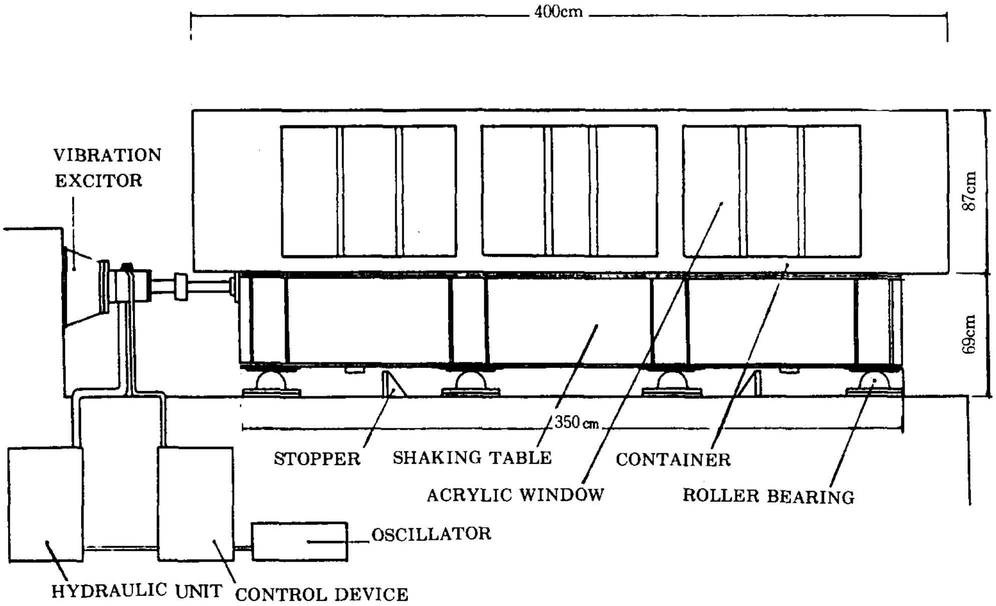


Fig. 2 The staking table unit at the University of Tokyo.

3. 2. Preparation of the model ground

The model structures were made in the form of double sheet pile wall and anchor sheet pile wall. Plastic plate, 3mm in thickness was used for the sheet pile. The plate was flexible enough to allow a large lateral deformation on the model structure during the shaking test. The distance between the two sheet piles of double sheet pile wall was varied in each test case. Sheet piles had a fixed boundary condition at the base, and the upper parts of the two sheet piles were connected by three tie-rods. The material of the tie - rod was bronze, 3mm in diameter.

The model ground was constructed by the method of water pluviation, in which the sand is rained slowly into the soil tank filled with water. The material of the ground was

Toyoura sand. It is the standard fine sand in Japan, and the material properties are written in Table 1. The size of the model ground was 55cm in depth and 400cm in length. The width was reduced to 30cm in order to save effort on preparing the ground.

The ground had two sand deposits, the liquefiable loose sand with relative density of around 40%, and the compacted dense sand with around 80%. For the purpose of observing lateral movements, several vertical lines were drawn by pouring the dark sand into the channels which had been already installed on the inner surface of soil tank window. These channels would be removed after completing the preparation of the model ground. Pore pressure meters and accelerometers were placed inside and on the surface of the ground to pick up the records. Also, some colored pins were inserted into the surface to observe surface movements.

Table 1. The properties of Toyoura sand

Specific Gravity, G_s	2.65
Maximum void ratio, e_{max}	0.974
Minimum void ratio, e_{min}	0.616
Uniformity, U	1.500
Effective grain size D_{10}(mm)	0.16

4. Test results and discussion

Six cases of model structures were tested through the shaking table. The types of model ground were decided to examine the effectiveness of the preventive countermeasure against flow failure due to liquefaction. These model structures can be classified into 3 types, such as:

(a) model embankment having no compacted zone,

(b) double sheet pile wall having a compacted zone, and

(c) anchor sheet pile wall having a compacted zone.

The input wave forms of shakings were given by a sine wave having the frequency of 3 or 5Hz and the acceleration of 120 or 200gals. The geometry and locations of accelerometers and pore pressure meters are shown in Fig 3. The ground conditions of the model structures are listed in Table 2. The shaking conditions and the final induced deformations of the tests are described in Table 3.

4. 1. Model embankments

Two model embankments with the slope of 1:2.75 were made for Case 1. One embankment(Case 1A) was shaken twice at the frequency of 5Hz and the acceleration of 120gals (Case 1A−1) and 200gals(Case 1A−2), respectively. The other(Case 1B) was shaken at 3Hz and 120gals. Figure 4 shows the acceleration and the pore pressure developments of

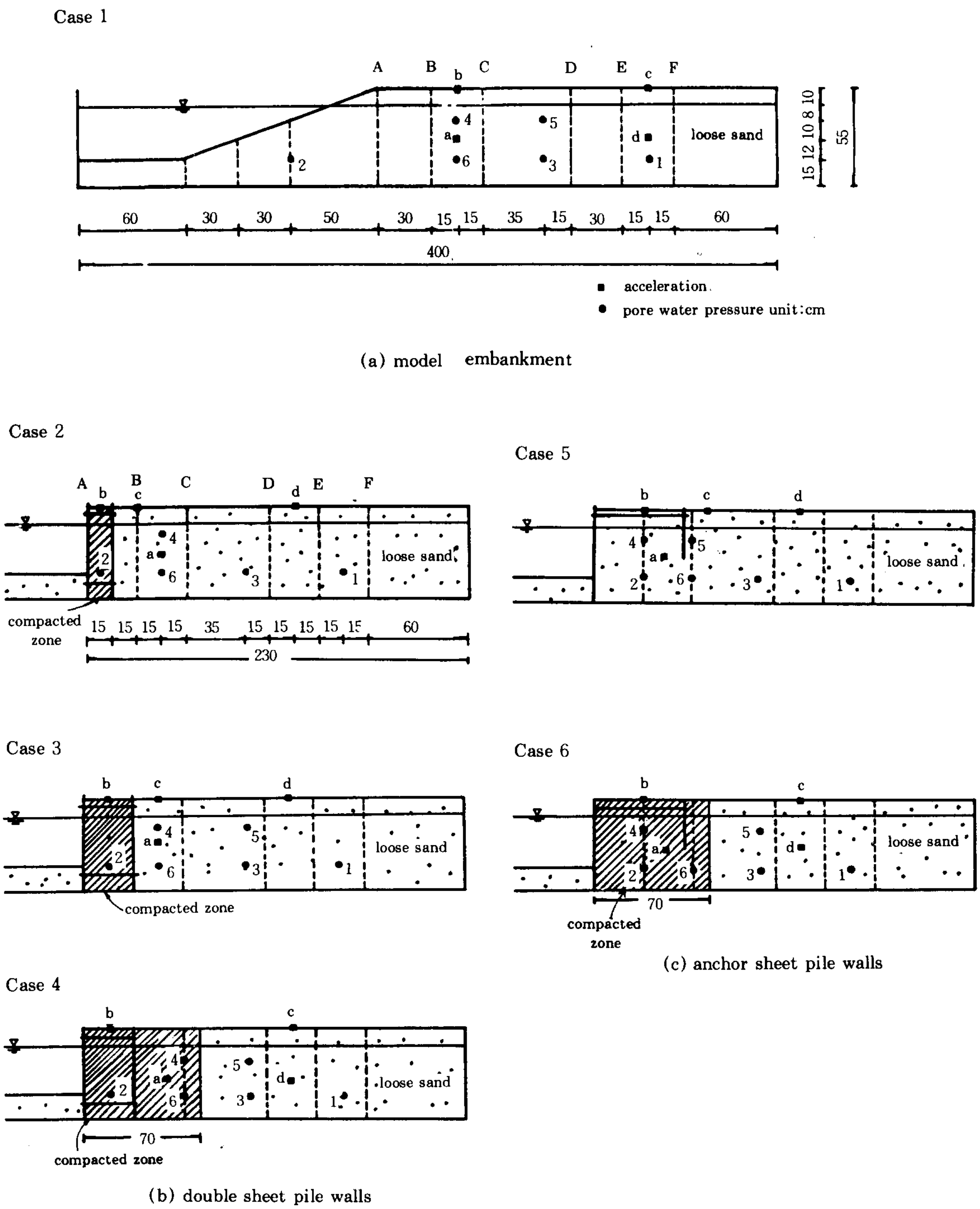


Fig. 3 Geometry of model waterfront structures

Table 2. Ground conditions of model structures

Case	Type	Length of compacted zone(cm)	Ground condition				Remarks
			loose ground		compacted ground		
			e	D (%)	e	D (%)	
1A	embankment	–	0.866	24.5	–	–	* D.S.P:
2B	embankment	–	0.838	32.9	–	–	double sheet
2	D.S.P	15	0.846	34.2	0.715	73.6	pile wall
3	D.S.P	30	0.818	42.6	0.678	84.7	
4	D.S.P	70	0.840	36.0	0.685	82.6	* A.S.P:
5	A.S.P	–	0.842	35.4	–	–	anchor sheet
6	A.S.P	70	0.823	41.1	0.681	83.8	pile wall

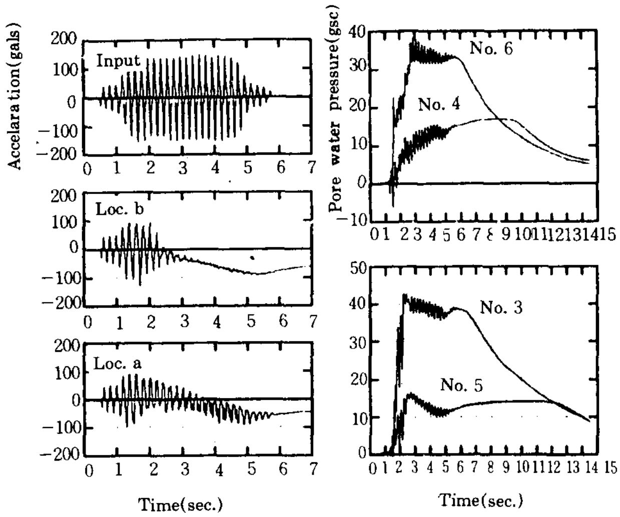
Table 3. Final induced deformation on the model structures

Case	Shaking		Measured deformations(cm)						Set. (cm)	Remarks
	Acc. (gals)	Time (sec)	A	B	C	D	E	F		
1A−1	120	6	10.0	10.0	6.0	5.0	5.0	2.5	4.0	
1A−2	200	15	–	41.0	26.0	17.5	15.0	8.5	10.0	
1B	120	42	20.0	14.0	11.0	7.0	9.0	2.0	10.0	
2	120	28	13.5	7.0	5.0	2.0	0.	0.	5.0	stable
3−1	120	25	5.0	5.0	2.0	0.	0.	0.	3.0	stable
3−2	200	35	0.	14.5	9.0	4.2	3.0	0.	6.0	stable
4−1	120	25	0.	0.	0.	0.	0.	0.	0.	
4−2	200	30	0.	0.	0.	−1.5	−1.5	0.	1.0	stable
4−3	400	30	7.0	4.5	1.5	−6.0	−3.0	−5.0	5.0	stable
5	120	22	2.0	18.7	9.0	8.0	4.0	3.0	4.0	unstable
6−1	120	26	1.2	0.	0.	0.	0.	0.	2.4	stable
6−2	200	30	3.0	2.0	2.0	−4.0	−5.5	−5.5	5.0	stable
6−3	400	23	9.0	2.0	−2.0	−7.0	−6.0	−8.0	7.0	stable

Note : For the frequencies of the shaking, Case 1A has 5 Hz, and the rest have 3 Hz

Case 1A−1 and Figure 5 shows the patterns of the deformation for Case 1. Brief descriptions of the test results are as follows.

The pore pressure build-up and dissipation show different patterns depending on the elevation of the ground. According to the records of pore water pressure developments in Case 1A−1 as shown in Fig. 4(b), at the lower points of Nos. 6, 3 and 1, the pore water pressures have built up suddenly and trigger the liquefaction. Then, they begin to drop gradually when the shaking has finished. On the other hand, at the upper points of Nos. 4 and 5, the developing pore water pressures have remained for quite a long time even after the termination of the shaking. Two lines of pore pressure development at the upper and



(a) Records of acceleration (b) Records of pore water pressure

Fig. 4 Records of accelerations and pore pressure in Case 1A−1

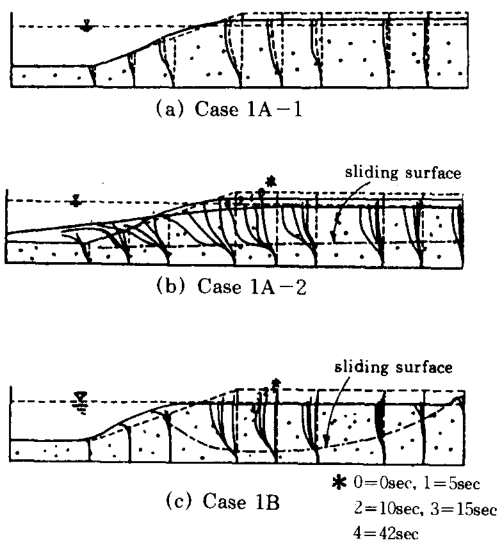


Fig. 5 Final flow deformation of model embankments

lower points have merged during the pore pressure dissipation. From this pattern, it can be noticeable that the reconsolidation of the liquefied ground starts from the bottom. The flow slide might have continued until the reconsolidation has completed. This seems to be the reason why a little progressive deformation was observed in Case 1A−1 just after the shaking. This kind of progressive deformation has been known to occur on the actual liquefied field due to seismic shaking.

In the second shaking of Case 1A, larger deformation was observed on most parts of the embankment, particularly at the location near the slope as shown in Fig. 5(b). The crest of the embankment was pushed downward and piled up at the downhill. Furthermore, because the elevation of the water table had risen during shaking, the unsaturated deposit above the water table lost its shear strength and sank into water. Most parts of the deposit except the area near the right boundary experienced liquefaction in Case 1B as shown in Fig. 5(c). The occurrence of the unliquefied zone seems to be caused by the boundary wall effect. There is a distinct difference in the mode of deformation between the liquefied and the unliquefied deposits. In the unliquefied zone, no considerable deformation appears, and a tension crack is generated on the surface of the boundary between two zones. Tension cracks have been observed on fields hit by earthquakes.

The records of pore water pressure build-ups show that the model embankments have experienced large flow type deformation at maximum pore water pressure ratios less than 100 percent. Initial shear stresses existing in the embankment accelerate the triggering of liquefaction. Actually, the ground near the slope was pushed away by sliding at the pore water pressure ratio between 0.7 and 0.8. As far as the size of the deformation is concerned, the flow type of deformation on the embankment seems to be influenced by both the geometry and ground conditions. For Case 1B, the duration of shaking is much longer than in Case 1A, but the ground deformations of Case 1B appeared to be smaller than those of Case 1A. Three possible reasons can be thought such that (1) the model embankment of Case 1A has a relative density smaller than that of Case 1B, (2) the liquefied area of Case 1A is larger than that of Case 1B and (3) Case 1A has a longer period of time in which the embankment has remained in the state of liquefaction. For a simple analysis of stability, a sliding surface can be assumed within the liquefied layer of the embankment. According to the histories of deformation in Fig. 5, sliding surfaces clearly appear in the ground near the slope. Being compared with the sliding surface of unliquefied slope in a conventional slope stability problem, the sliding surface spreads over a wide extent of the liquefied layer.

4. 2. Double sheet pile and anchor sheet pile walls

Three cases of double sheet pile walls were tested for Cases 2, 3 and 4. The model grounds were made of two zones: the compacted dense sand zone between two piles and the liquefiable loose sand zone. These two zones behaved independently during the tests. Rather than sudden build-ups of pore pressures as appeared in loose sand zones, the negative pore water pressures dev loped in the compacted zone due to the dilatency. Figure 6 show the final deformations of the double sheet pile walls.

The structure was quite stable in Case 2. As the shaking continued, the lateral deformation had almost stopped after spreading to some extent. However, quite a lateral deformation is induced on the sheet pile structure. The wall was pushed by the lateral forces of the liquefied ground. Most deformation occurred in the area behind the wall even though the whole loose sand deposit was liquefied. Since the sheet piles have fixed conditions at the bottom, only a rotation is allowed. The deformation of the ground is directly influenced by the rotational resistance of the sheet pile wall.

A little deformation appeared on the structure at the first shaking of Case 3(Case 3−1) owing to the enlarged zone of compaction. However, after the second shaking(Case 3−2), large deformation showed in the wall. The model structure of Case 4 behaved very strongly against the liquefied ground. At the first shaking(Case 4−1), the whole ground did not liquefy. During the second shaking(Case 4−2), the loose sand deposit liquefied, but only a little settlement appeared on the surface. At the third shaking(Case 4−3), a bulge appeared on the front wall and the lateral deformation of the liquefied ground directed toward the opposite of the waterfront. The unsaturated layer above the water table was pushed toward the opposite direction because of the elevation gradient caused by the difference between the settlement of the dense and loose deposits.

The model anchor sheet pile walls consist of Cases 5 and 6. The anchor plate was embedded in loose snad deposit in Case 5, so the flow type of failure occurred on the structure.

The anchor sheet pile wall deformed continuously until the end of shaking. Hence, three tests of Case 6 showed that the compacted zones in anchor sheet pile wall resisted very strongly the lateral pressure of liquefied ground. The behaviors of the deformation were similar to those of Case 4. Figure 7 shows the final deformations of the anchor sheet pile walls.

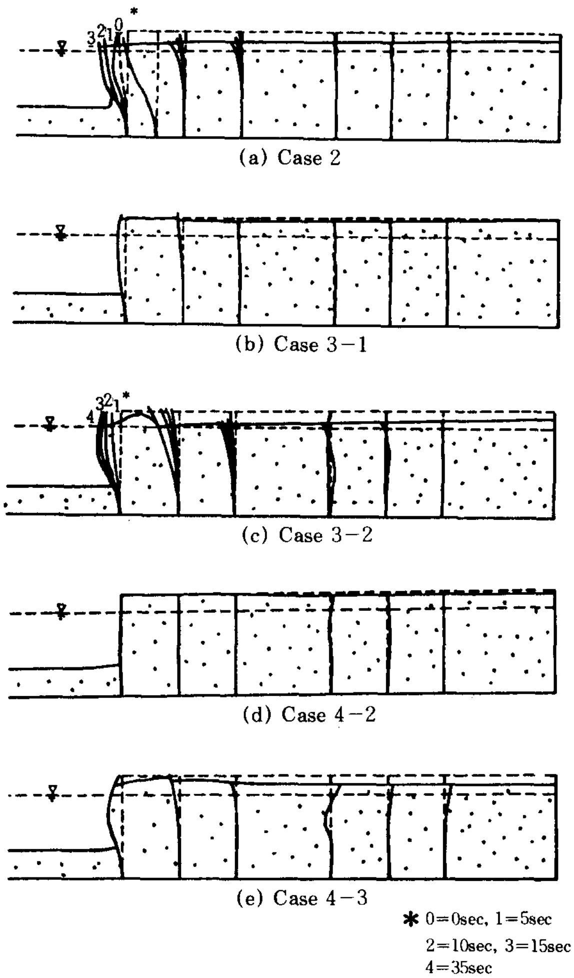


Fig. 6 Final flow deformation of the double sheet pile walls

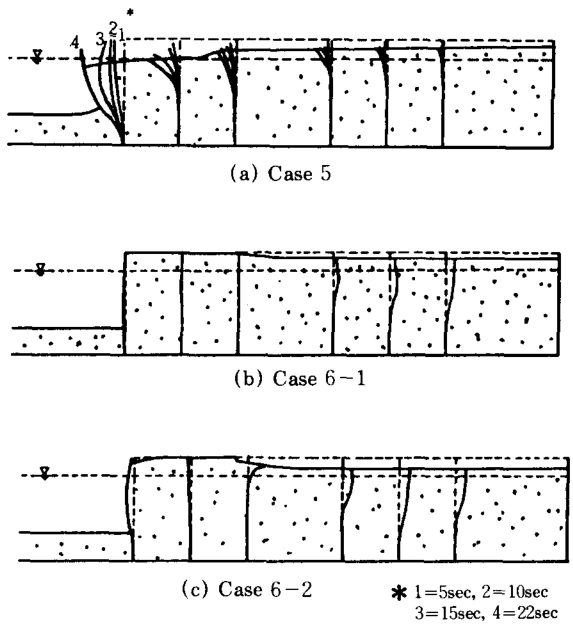
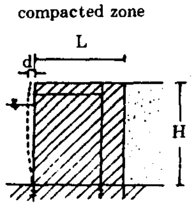


Fig. 7 Final flow deformation of the anchor sheet pile walls

4. 3. The effects of the compacted zone in the waterfront structures

The shaking table tests of the model structure reveal that the compacted zone plays a very important role to prevent large deformations both on the structure and the liquefied ground. The measured deformations from the tests can be summarized in terms of the extent of the compacted zone and the magnitude of the input acceleration. Figure 8 shows the case in which only a limited deformation was observed on the structure. The deformation ratio, d/H represents the maximum wall movement divided by the height of the wall from the fixed end. On the other hand, the reinforced ratio, L/H is the ratio of the width of the compacted zone to the height of the sheet pile wall. Actually, the results include the interplayed resistance of the sheet pile itself and the densified sand. However, since the model sheet pile was made of very flexible material, the resistance of the sheet pile could be neglected.

If an allowable deformation ratio is assumed to be 0.025, the reinforced ratio, L/H is obtained to about 1 for an acceleration of 120gals, and to about 1.5 for 200gals from Fig. 8. It means that a double sheet pile wall requires a co mpacted zone as wide as the height of the sheet pile wall to be safe from the lateral pressure of the liquefied ground behind the structure. For the practical application in design, the relationship in Fig. 8 has some limitations, because (1) the strength of the sheet pile itself and the effects of scale are not considered, (2) the test data are not sufficient and (3) the fixed end conditions of sheet piles are not guaranteed in the actual field. However, it provides very good quantitative information on the extent of the reinforcement.



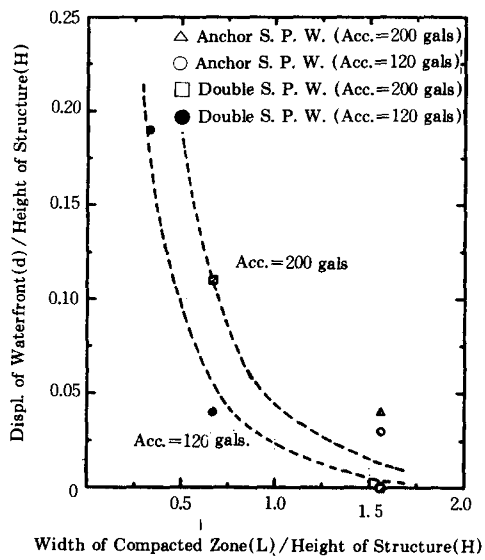


Fig. 8 The displacement of the waterfront structure according to the width of compacted zone.

5. Summary and Conclusion

The characteristics of the liquefied ground and the effects of a compacted zone were examined for the purpose of protecting the waterfront structure from flow failure. Throughout the shaking table tests on embankments, double sheet pile walls, and anchor sheet pile walls, some conclusions can be found as follows:

(1) The pore pressure ratio triggering the flow type of failure on the slope depends on the magnitude of the initial static shear stress. Pore pressure ratio over 0.8 seems to be critical to the stability of the slope.

(2) For a simplified stability analysis, a sliding surface can be assumed to exist to a very wide extent within the liquefied layer.

(3) The amount of the deformations in the liquefied embankment is affected by the density of the ground, the area of liquefied ground, the depth of the liquefied zone, the surcharge of the unliquefied layer, and the period of time at which the ground has remained in a state of liquefaction.

(4) The amount of deformation on the waterfront structure depends on the extent of the compacted zone and the magnitude of the input acceleration.

(5) As to the extent of the compacted zone, an area of at least 1−1.5 times to height is required to protect the waterfront structure from excessive deformation.

Therefore, the results of the skaking table tests provide good quantitative information on the behavior of flow failure and the extent of reinforcement for protecting the waterfront structure from flow failure. However, for the better practical countermeasure against liquefaction, further research must be done on areas such as the behavior of the interaction among the liquefied ground, the compacted zone and the sheet pile structure.

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