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Experimental study for liquefied soil in a gap between underground walls

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ABSTRACT: As the steel or concrete walls constructed underground can prevent liquefied soil from flowing, they have been used as the effective anti-liquefaction countermeasures. However, it is difficult that these countermeasures are constructed perfectly under a ground which has buried objects or rocks, so the walls may have some tiny gaps. As the behaviour of liquified soil flowed out of a gap is not clarified. Therefore, the author carried out two steps of experiments. Firstly, the author focused on a small gap between the walls and carried out shaking model tests. Secondly, the author investigated the effect of the gaps on a whole construction. Shaking model tests for an embankment reinforced by the sheet-piles which has gaps, or no gaps were performed. Through the second model tests, it was confirmed that the soil was pushed from the gaps between sheet-piles, which induced the settlement of the embankment.

1 INTRODUCTION

The severe seismic damage to constructions in Japan's Tohoku region was caused by the 2011 off the Pacific Coast of Tohoku Earthquake. A large earthquake such as Nankai Trough Earthquake is also concerned to occur in the near future and there is a fear that construction or residential areas will sink by liquefaction. As the steel or concrete walls constructed underground can prevent liquefied soil from flowing, they have been used as the effective anti-liquefaction countermeasures (Fujiwara, Koseki, Otsushi & Nakayama 2013). However, it is difficult that these countermeasures are constructed perfectly under a ground which has buried objects or rocks, so the walls may have some tiny gaps. In addition, there are also some methods originally designed to be with gaps such as the PFS (Partial Floating Sheet-pile) method (Fujiwara, Nakai & Ogawa 2019). As the behaviour of liquified soils flowed out of a gap is not clarified, the effect of the gaps on a whole construction is also not clarified. Therefore, the author carried out two steps of experiments. Firstly, the author focused on a small gap between the walls and carried out shaking model tests to investigate the liquified soil behaviour. Secondly, the author investigated the effect of the gaps on a whole construction. Shaking model tests for an embankment reinforced by the sheet-piles which has gaps or not were performed. Through the two tests, a possibility of modelling structures with gaps in the 2D numerical analysis was discussed.

2 SOIL BEHAVIOUR BETWEEN GAPS

2.1 *Test condition*

2.1.1 *Device*

A test box shown in Figure 1 was used for "the wall gap test". The rigid box was used in this study. The dimension of box is 353 mm of width, 170 mm of height and 200 mm of depth. These dimensions are approximately 1/25 of actual size. The box was fixed on a table of the shaking device during the motion.

2.1.2 *Test cases*

Three cases shown Table 1 were carried out to confirm the behavior of liquefied sand in a small gap. Case-A (without walls) was also carried out in order to compare to Case-B and C (with walls).

2.1.3 *Procedure*

The procedure of the test is explained in numerical order;

(1) Two wooden walls were put in the box with a gap. The bottom of the walls was fixed to the bottom of the test box by adhesives. The walls have enough thickness (13 mm) to be considered as rigid body.
(2) Water and Toyoura sand were put in the box in this order. Toyoura sand with the characters of soil density (ρ_s=2.64 g/cm^3), maximum void ratio (e_{max}=0.927), minimum void ratio (e_{min}= 0.635), permeability coefficient (k=1.94×10^{-2} cm/s

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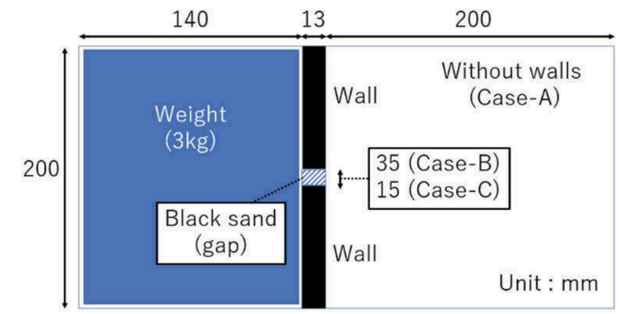
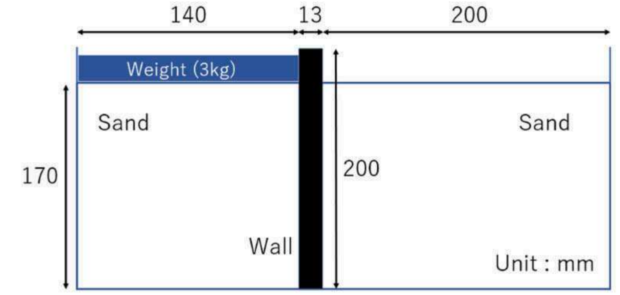


(a) Plain figure



(b) Cross section

Figure 1. Test box.

Table 1. Test cases.

	Type	Gap (mm)	Dr (%)
Case-A	Without walls	-	50
Case-B	Two walls	35	55
Case-C	Two walls	15	59

(D_r=45%)) and equal factor (U_c=1.37), 50% diameter (D_{50}=0.325 mm), was used as liquefiable soil.

(3) Black sand that the same sand as (2) colored with black ink was put between the two walls, which indicated the behaviour of liquefied soil. In advance, the authors put thin plastic sheets to enclose the gap in order to make the black sand form. Then the authors put the black sand in the enclosed area, after that pulled the sheets out softly.

(4) The saturated sand layer which had a thickness of 170 mm and relative density (Dr) of 50 % approximately was made.

(5) A weight which was gravels wrapped with a plastic sheet was put on one side of the soil surface. The mass of the weight was 3 kg which was equivalent to a common embankment for this model size.

(6) After the model was put on the shaking table, a sine wave motion was given to the shaking table. The sine wave had a frequency of 5 Hz, a duration of 5 seconds and a maximum amplitude of 5 m/s^2, shown in Figure 2. This condition can be converted to 0.27Hz, 56 seconds and 5 m/s^2 in actual size based on the similarity rule (Iai 1988), which had an extremely large motion.

(7) After shaking, the authors absorbed the extra surface water by a sponge, took off the weight and dug the sand to measure the horizontal displacement of the black sand.

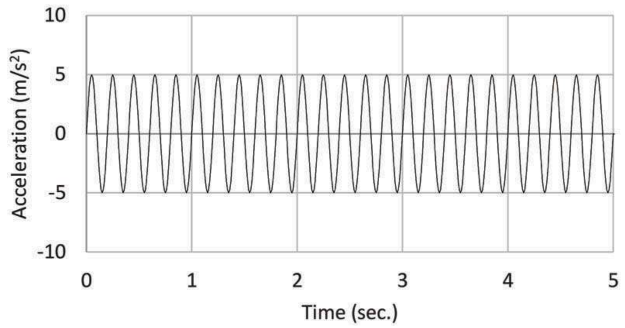


Figure 2. Input motion.

2.2 *Test result*

Figure 3(a) shows a picture of the surface after shaking for Case-B as an example of test results. Almost whole ground was liquefied by the large motion. The weight was already taken off in Figure 3(a). As the sand was liquefied by the motion, the sand under the weight was settled shown, which moved the black sand laterally. The settlements at the side of the gap under the weight were almost the same as Case-A (40 mm), Case-B (38 mm) and Case-C (35 mm).

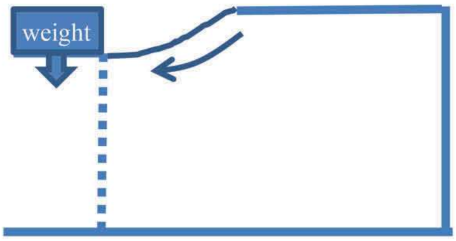
The authors dug the sand and took pictures at 10mm, 25mm, 50mm, 75mm and 100mm under from the surface. Figure 3(b) shows a picture of 10 mm under the sand surface for Case-B as a typical picture. The black sand was pushed toward the right side by the weight, which had an arc shape.

Figure 4 shows the residual horizontal displacement of the black sand along the vertical direction. The horizontal displacement of the black sand is defined as the longest distance from the gap to the outside of the black sand arc. As the black sand was mixed with non-colored sand, it was difficult to be observed clearly. Therefore, these values were evaluated in 5 mm unit.

As for Figure 4, the residual horizontal displacement became larger as the width of the gap became larger under the depth of 25 mm (Case-A is considered to have an infinity gap). The horizontal displacement of Case-B was close to that of Case-A. The horizontal displacement of Case-C which had the smallest values was approximately 25 % less than that of Case-A. On the other hand, Case-A had the smallest horizontal displacement over the depth of 25 mm. It is considered that this is because, although the surface sand was pulled back to the weight for Case-A, shown as Figure 3(c), the walls prevented the sand from pulling toward the weight for Case-B and C.

(a) Surface ground

(b) 10 mm under the surface



(c) Conceptual figure of soil back for Case-A

Figure 3. Test results.

3 CONSTRUCTION BEHAVIOUR WITH GAPS

The authors focused on an embankment reinforced by the sheet-piles as a construction which has gaps inside it. Sheet-piles are installed into the ground and lined in the depth direction along the toe of an embankment occasionally, which reinforce an embankment against earthquake (Fujiwara, Taenaka, Otsushi, Yashima, Sawada, Hara, Ogawa, & Takeda 2017). As a specific type of the reinforcement, the different lengths of sheet-piles are installed alternately, called the PFS (Partial Floating Sheet-pile) method shown in Figure 5 (Fujiwara, Nakai & Ogawa 2019). In this chapter, the authors focused on the reinforcement with the sheet-pile skipping alternately, which made a gap between two sheet-piles. The behaviour of the embankment with and without sheet-piles gaps is discussed by a series of shaking model tests.

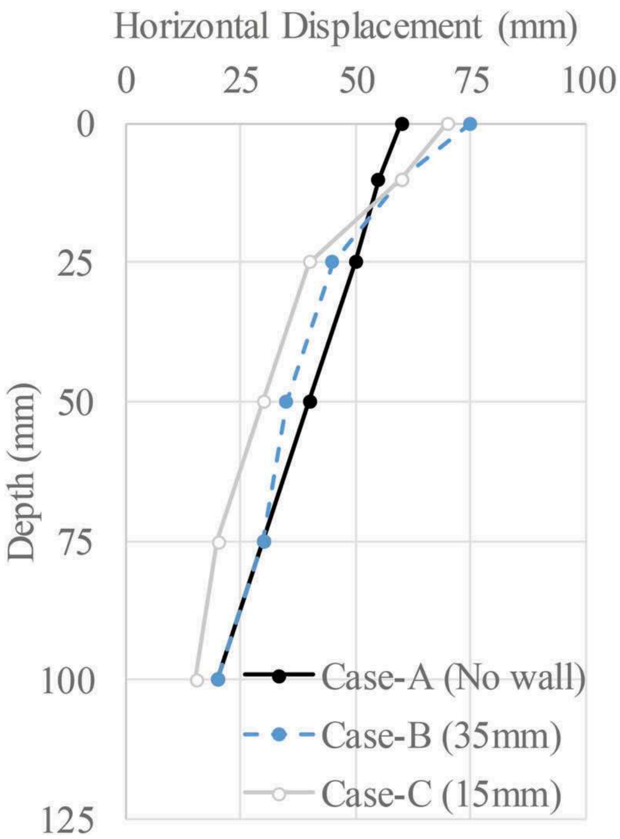


Figure 4. Horizontal displacement.

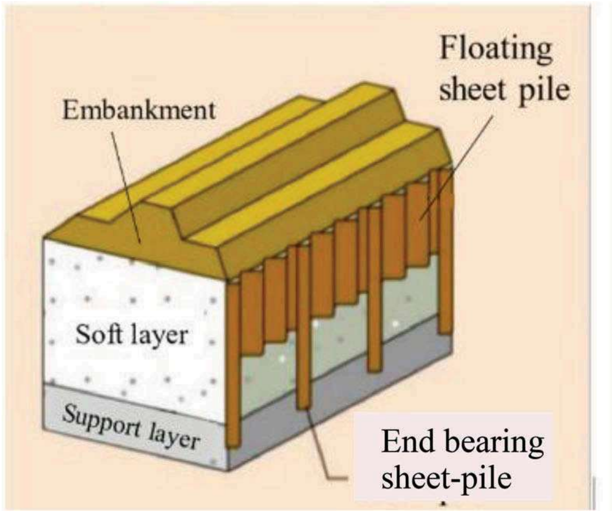


Figure 5. An example of construction with gaps (the PFS method).

3.1 *Test condition*

3.1.1 *Dimension*

A rigid container was used for the tests. The dimensions were set as approximately 1/20~1/30 of actual size shown in Figure 6. The tests were conducted under 1 g gravity field. The target embankment was cut in half size considering symmetry for the

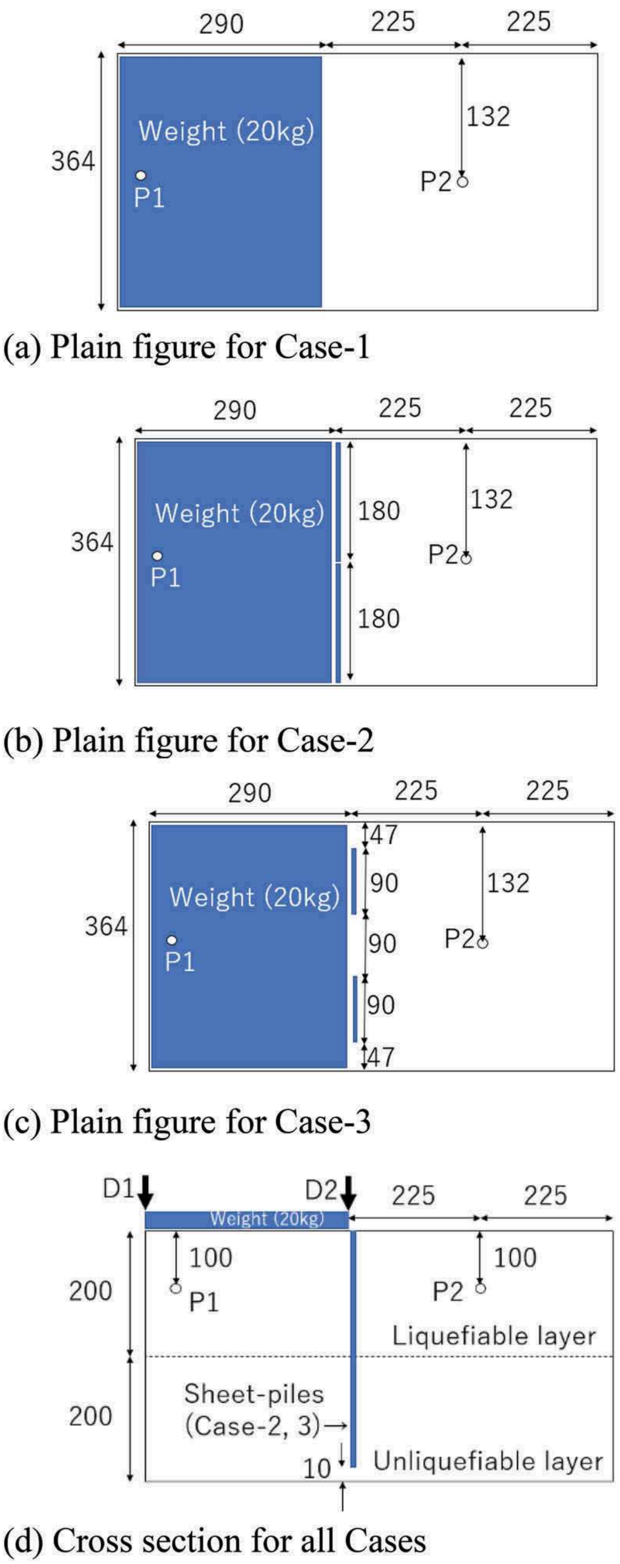


Figure 6. Dimensions of test cases.

limitation of the container's size. A steel plate was used for the test as an embankment, mentioned later.

3.1.2 *Soil character*
The test ground was made of Toyoura sand same as used in Chapter 2. The relative density and weight per unit volume for the liquefiable layer are indicated in Table 2.

Table 2. Test cases (liquefiable layer).

	Case-1	Case-2	Case-3
	No sheet-piles	Sheet-piles (without gaps)	Sheet-piles (with gaps)
γt (kN/m^3)	18.7	18.7	18.6
D_r (%)	41	44	38

3.1.3 *Procedure*
The test ground was made by putting water after soil. Firstly, the unliquefiable layer which had a thickness of 200 mm and relative density (D_r = 95 %) was made by sticking with a wood bar. Secondly, model sheet-piles made from acryl which had a thickness of 5 mm, a length of 400 mm and Young's ratio 3.2 MPa, was installed 190 mm into the unliquefiable layer. The thickness and the material are decided considering the similarity rule against the actual size (Iai 1988). Thirdly, the liquefiable layer which had a thickness of 200 mm and relative density (D_r = 40 %) was made by sprinkling the sand. Finally, a steel plate was used instead of an embankment. The plate which had a size of 360 x 290 x 20 mm and a weight of 20 kg equivalent to the embankment was put on one side of the surface ground. The steel plate and model sheet-piles were put with 1~10 mm clearance from the side of the container not to be attached each other.

3.1.4 *Test cases*
Three cases were conducted as follows;
Case-1: Without sheet-piles
Case-2: Sheet-piles without gaps
Case-3: Sheet-piles with gaps
Two sheet-piles whose width was 180 mm were used for Case-2 and 3. There was a gap of 90 mm equivalent to approximately 2 m in actual size for Case-3.

3.1.5 *Measurement points*
The measurement points are also indicated in Figure 6. The vertical displacements of the steel plate were measured at D1 and D2 for Figure 6(d). The excess pore water pressures were measured at P1 and P2 100 mm under the surface. The bending strain of the sheet-piles were measured at four points; 100 mm, 150 mm, 200 mm and 260 mm from the bottom of the container respectively.

3.1.6 *Input motion*
A sine wave which has a frequency of 3 Hz, duration of 5 seconds and a maximum acceleration of 3.0 m/s^2 shown in Figure 7, was given to the bottom of the test container.

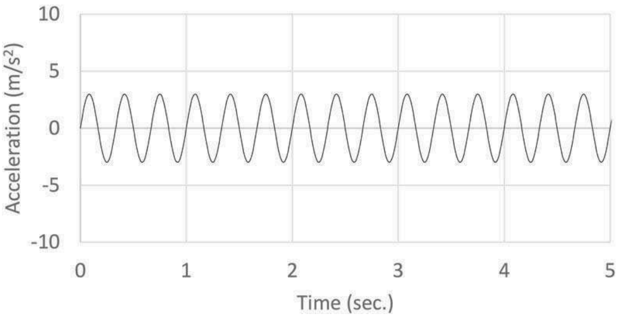


Figure 7. Input motion.

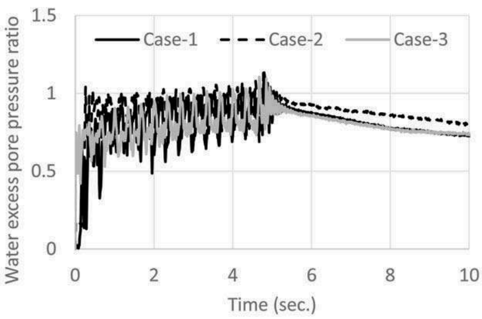
3.2 *Test result*

3.2.1 *Excess pore water pressure ratio*

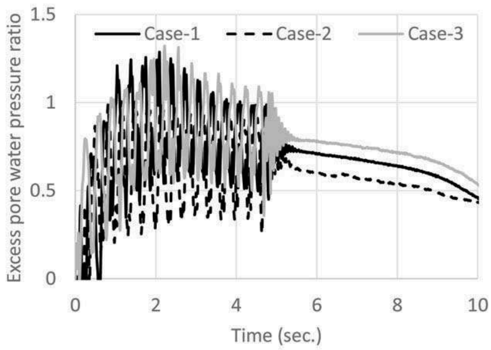
The time histories of excess pore water pressure ratio at P1 and P2 are shown in Figure 8. The excess pore water pressure ratio means the value that the excess pore water pressure is divided by the initial effective stress, calculated from the weight and the unit weight of the soil. The excess pore water pressure ratio at P1 and P2 closed to 1.0 during shaking, which indicated that the ground was liquefied during shaking. The excess pore water pressure ratio was reduced as time passed.

3.2.2 *Settlement*

The time histories of vertical displacement at D1 and D2 are shown in Figure 9. The weight plate was settled down during liquefaction for all cases. As the settlement at D1 was different from that at D2, the plate was inclined. The residual average values of D1 and D2 were 56mm (Case-1), 34mm (Case-2) and

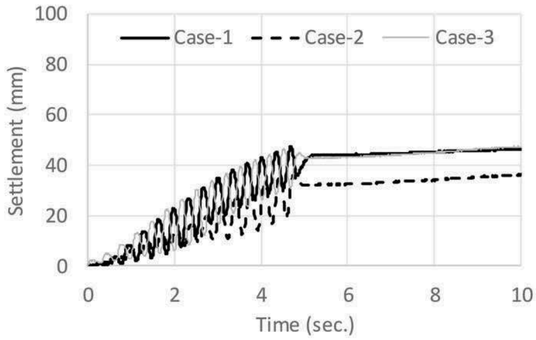


(a) P1

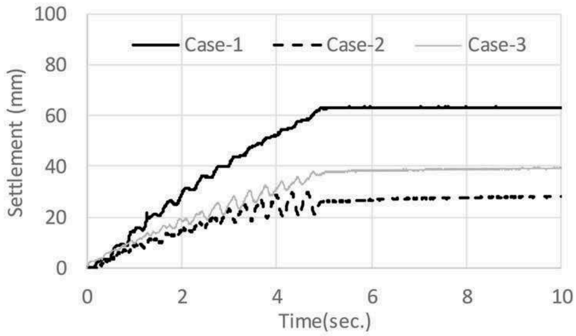


(b) P2

Figure 8. Excess pore water pressure ratio.



(a) D1



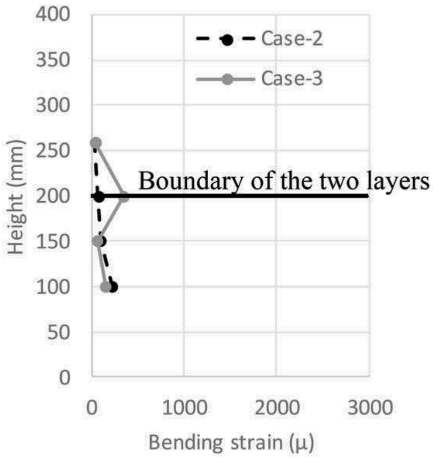
(b) D2

Figure 9. Weight settlement.

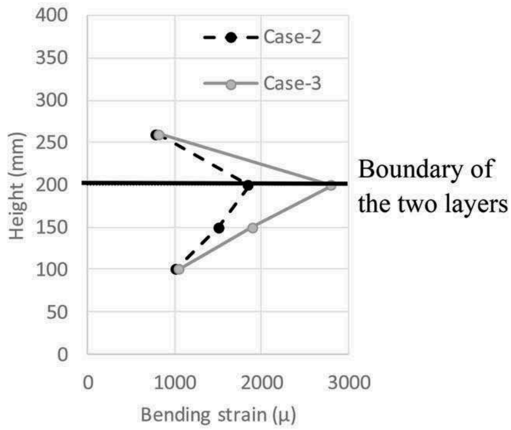
45mm (Case-3) respectively, which indicated that sheet-pile with or without gaps could reduce the settlement. The ground was pushed laterally by the weight of the plate during liquefaction, which caused the settlement. The sheet-pile installed next to the weight prevented the ground from moving, which reduced the weight settlement accordingly. Although the settlement at D2 is larger than that at D1 for Case-1, the settlement at D1 is larger than that at D2 for Case-2 and 3. It is assumed that the settlement at D2 was specially reduced by the near sheet-pile. The average values of D1 and D2 for Case-3 was larger than that for Case-2. It is considered that there are two reasons. One reason is that the soil passed through the gap for Case-3. As this width of 90 mm was much larger than 35 mm, the soil passed through the same as without sheet-piles, as discussed in chapter 2. The other reason is that the deformation of the sheet-pile for Case-3 was larger than that for Case-2, to be discussed in the next section.

3.2.3 *Bending strain*

The residual and maximum bending strain are shown in Figure 10. The sheet-pile had bending strains caused by the ground moving laterally. The bending strain in Case-3 was larger than that in Case-2. It is considered that more comprehensive earth pressure applied to the sheet-pile per width, which made the weight settle more in Case-3. In addition, although the width of the sheet-piles in Case-3 was half of that in Case-2, the bending strain in Case-3 was not



(a) Residual



(b) Maximum

Figure 10. Bending strain.

half of that in Case-2 exactly. The relationship of the earth pressure with the width of sheet-piles should be discussed. It is assumed that the bending of the walls makes the lateral displacement of soils large.

4 DISCUSSION FOR NUMERICAL ANALYSIS

Through the two model tests, the way of numerical analysis applied to a construction with gaps is discussed in this chapter. As the structure has gaps, there are multiple sections in the depth direction. Therefore, the 3D numerical analysis is more suitable to analyze than the 2D analysis to gain a precise result (Fujiwara, Nakai & Ogawa 2021). On the other hand, the 2D numerical analysis is more suitable in design work for the easiness and time-shortening. Generally, the rigidity of sheet-piles is adjusted to be averaged value in the depth direction when a 3D model is converted to a 2D model. However, there should be a limitation of this way because soil passes through gaps and the bending deformation of the sheet-piles are different in the depth direction, as indicated in chapter 2 and 3. The limitation for numerical analysis should be discussed by the comparison of 3D and 2D methods in the future.

5 CONCLUSIONS

The authors carried out two model tests for liquefaction; one was the wall gap test which had different width of gaps to investigate the soil behaviour between walls, the other was the embankment test which had sheet-piles with or without gaps at the toe of it with and without gaps to investigate the effect of gaps on a whole construction. From the above, following results were gained.

1) Through the wall gap test, the wider the width of the gaps became, the larger the lateral displacement of the liquefied soil between gaps became. The lateral displacement of soil in the width of 35 mm gap was close to that without wall considered to be the same as the infinity width.
2) Through the embankment test, the soil was pushed laterally by the weight during liquefaction. The effectiveness of sheet-piles against settlement can be confirmed not only for the case without a gap but also the case with a gap. Additionally, it was confirmed that the sheet-piles with a gap had more earth pressure per width than that of without a gap per depth.
3) After the two tests, the application of numerical analysis for a structure with gaps was also discussed. It is considered that there should be a limitation when a 3D numerical model is converted into a 2D numerical model, because soil passes through gaps and the bending deformation of the sheet-piles are different in the depth direction. The limitation for a numerical analysis should be discussed by the comparison of the 3D and the 2D method in the future.

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