STUDY ON SHEET PILE WALL METHOD AS A REMEDIATION AGAINST LIQUEFACTION

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SUMMARY

Several kinds of treatment methods have been developed as the liquefaction countermeasure. However, only a few methods are practical for existing structures especially in urban areas. One of them is the steel sheet pile walls method which is applicable under limited treatment site and work space. The authors have examined the applicability of that method to the countermeasure for various types of structures. In this paper, liquefaction countermeasures with steel sheet pile walls for some kinds of structures are introduced and effectiveness of them is demonstrated through model tests and finite element analyses. Following results were obtained. (1)Results of finite element analyses indicated that the settlement of embankments due to liquefaction could be reduced by the installation of a pair of self-standing sheet pile walls at the toes of it. (2) The effect of enclosing the foundation ground with the sheet pile wall as a countermeasure for tanks was confirmed in model test results, particularly for the steel sheet piles with drain capability. (3)The steel sheet pile with drain capability was especially effective for underground structures. (4)As a countermeasure for pile foundations against lateral flow of liquefied sand behind quay walls, the sheet pile wall was effective which was installed at the lower side of the pile foundation.

INTRODUCTION

Liquefaction of sand deposit often damaged various kinds of structures. During 1995 Hyogoken-nanbu earthquake, river dikes and embankment of irrigation ponds severely settled and collapsed due to liquefaction of foundation soil. Lateral flow of liquefied soil behind quay walls also damaged pile foundations. Several treatment methods represented by some kinds of soil improvement technique have been developed as the liquefaction countermeasure. However, only a few methods are practical for existing structures because of confined construction site especially in urban areas. One of them is the steel sheet pile walls method which is applicable under limited treatment site and work space. Then, the authors have examined the applicability of the method to liquefaction countermeasure for various types of structures. In this paper, liquefaction countermeasures with steel sheet pile walls for some kinds of structures are introduced and effectiveness of them is demonstrated through model tests and finite element analyses. First, the countermeasure for embankments on the liquefiable foundation are investigated through finite element analyses. Secondly, the effectiveness of the sheet pile method for outdoor tanks on the liquefiable foundation are examined through shaking table tests. In these tests steel sheet piles equipped with a channel with a number of holes to drain pore water are applied in addition to normal sheet piles. Next, the experimental results are introduced on the effectiveness of the sheet pile wall method including that with drain capability for underground structures. Finally, results of many shaking table tests are demonstrated to specify the effective treatment method for pile foundations against lateral flow of liquefied soil behind quay walls.

COUNTERMEASURE FOR EMBANKMENTS

Settlement and slope failure of soil embankment often result from spreading out of the liquefied soil underlying the embankment due to its weight. One of the liquefaction countermeasure for embankmentsts is the installation of steel sheet pile walls at the toes of them. Especially for railway and road embankments, pairs of sheet pile

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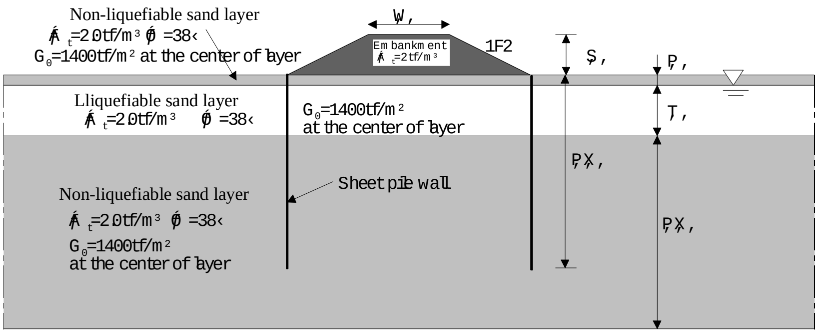
walls are driven at the toes of them and often connected with tie rods. For river dikes and embankments of irrigation ponds, however, self-standing type of the sheet pile wall is preferable because the installation of tie rods causes water leak. Therefore, authors have conducted many model tests to investigate some types of treatment method with steel sheet pile walls [Tanaka et al., 1996 ; Tanaka et al., 1996]. This paper introduces finite element analyses on effectiveness of the self-standing type sheet pile wall.

OUTLINE OF ANALYSIS

Parametric study on effectiveness of the steel sheet pile wall method were conducted for the model ground shown in Fig.1 under a variety of conditions given in Table 1. A finite element analysis program with effective stress method, ‘FLIP’ [Iai et al., 1990] , was applied to these analyses. The embankment of 4m height and 8m crown width overlies the foundation ground which consists of sand layers: the upper unsaturated sand layer of 1m thickness, the liquefiable sand layer of 5m thickness with liquefaction resistance shown in Fig.2 and the lower non-liquefiable layer. No friction was assumed between the soil and the sheet pile wall. This assumption is implemented by imposing the condition that the displacements of the nodal points of soil elements and the beam elements are the same with each other in the horizontal direction but can take independent values in the vertical direction. Input waves were given at the fixed boundary(relative displacement Ux=Uy=0) of the bottom of the analysis domain. The effects of free field motion, simulated by one dimensional response at outside fields, were taken into account by transmitting them through the viscous damper at the both sides of boundaries.



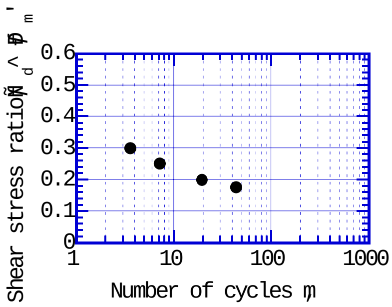


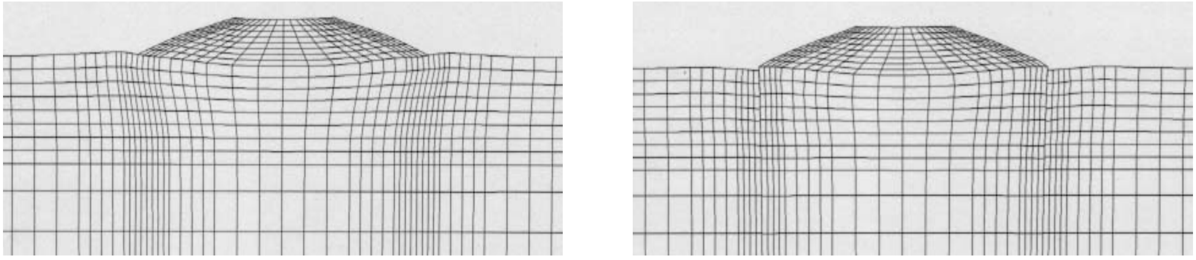
Figure 1: Cross section of model ground for finite element analyses **Figure 2: Liquefaction resistance of liquefiable layer**

Table 1: conditions of analyses

Input wave	Duration of input motion	Maximum acceleration of input motion	Condition of reinforcement
Sinusoidal	10sec,2Hz	100,150,250,400gal	no-reinforcment reinforcement by SP III reinforcement by SP V
Hachinohe EW	21.6sec	100,183,250,400gal	
Kobe NS	30.0sec	100,250,400gal	

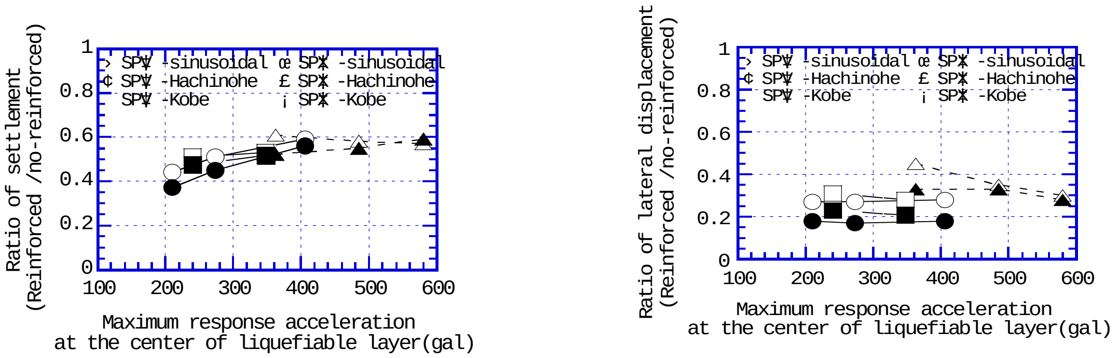
RESULTS OF ANALYSES

Figure 3 shows examples of the computed residual deformation of the embankment and the foundation ground. Figure3 (a) and (b) are those for the no-reinforced model and the reinforced model to which Hachinohe EW wave with the maximum acceleration amplitude of 183gal was applied. It can be found that lateral displacement of liquefied foundation soil is restricted by sheet pile walls, and then this effect results in reducing settlement of the embankment. Figure 4 (a) shows the relationship between the settlement ratio of the reinforced embankment to the no-reinforced embankment and the maximum response acceleration at the center of liquefiable layer and (b) shows that of the lateral displacement ratio under the toe of the embankment. It can be seen that the installation of sheet pile walls can reduce the settlement of the embankment to 40%~60% for the model ground shown in Fig.1. These results also indicate that the effectiveness of the sheet pile wall method tends to decline under the condition of severe liquefaction even though lateral displacement of ground under the toe of the embankment is restricted effectively. However a certain degree of the effect of the sheet pile wall method seems to be expected even under the severe condition of liquefaction.



(a) No-reinforced embankment (b)Reinforced embankment

Figure 3: Residual deformation of the embankment and the foundation ground



(a)Ratio of settlement of reinforced to no-reinforced embankment

(b)Ratio of lateral displacement of reinforced to no-reinforced embankment

Figure 4: Relationship between settlement of the embankment and response acceleration, and between lateral displacement under the toe of the embankment and response acceleration

COUNTERMEASURE FOR OUTDOOR TANKS

The harmful deformation of outdoor tanks and the breakdown in buried pipes around them are often induced by the settlement of tanks due to liquefaction of foundation ground. One of the countermeasure for existing tanks is to enclose the foundation ground under them with steel sheet piles [Zheng et al., 1996]. It is, however, still possible that the inclination and differential settlement is induced since the sheet pile enclosure can not necessarily prevent the soil under tanks from liquefaction. Then, the authors have carried out shaking table tests to confirm the effectiveness of the sheet pile enclosure and to examine the applicability of the special sheet pile with drain capability.

TEST PROCEDURE

Figure 5 shows the setup of the model tests and instrumentation for measuring the response. The conditions for tests can be found in Table 2. The models were contained in a rigid container of dimensions 2000mm long, 1000mm high and 1000mm wide. The dimension of the model was determined on the basis of the similarity low [Iai, 1988] as that was an approximate one-fifty geometrical scale model. Sand used for tests was Toyoura sand with a mean diameter of 0.18mm. To reduce the permeability considering the similarity low, the sand layers were saturated with the cellulose solution which was 30 times as viscous as water. The model ground consists of two sand layer. The upper one is a liquefiable layer of 250mm thickness and its unit weight was approximately 1.93gf/cm^3. The tank model of 318mm in diameter and 100mm in height is a rigid container which has the contact pressure of 20gf/cm^2. The sheet pile model is a steel plate of 0.6mm thickness formed into a circle. In case of that with drain capability, rectangular vertical drain pipes with the cross section of 15mm x 10mm are equipped on the both surfaces of steel plate at intervals of 80mm. Each model was shaken by 50 cycles of horizontal sinusoidal motion at frequency 5Hz and at the acceleration levels of 150, 200 and 300gal. One level of input acceleration was applied during one test run and this was repeated for each level of acceleration.

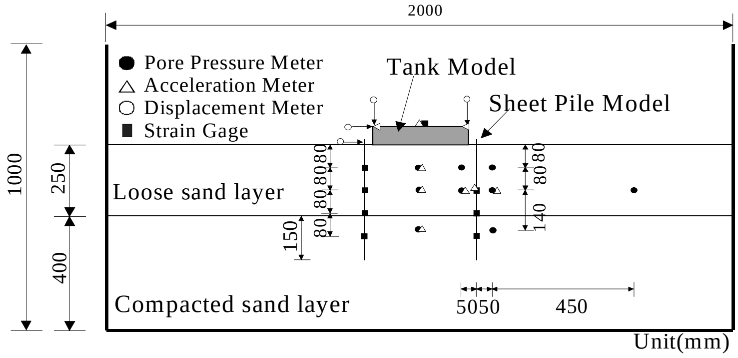


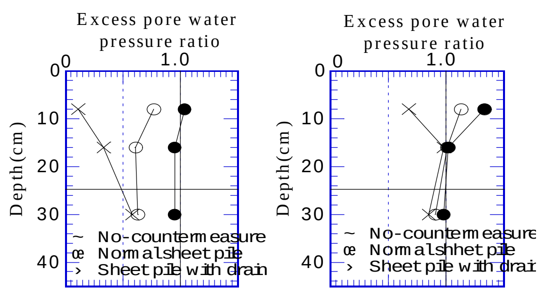
Figure 5: Cross section of model used in shaking table tests on the countermeasure for the tank

Table 2: Conditions of shaking table tests

Type of countermeasure	Dr of loose layer	condition of input motion	
No-countermeasure	71.7%	153, 214, 308gal	5Hz Sinusoidal 10sec
Enclosure with normal sheet pile	70.8%	157, 212, 314gal	
Enclosure with sheet pile with drain	62.1%	155, 211, 306gal	

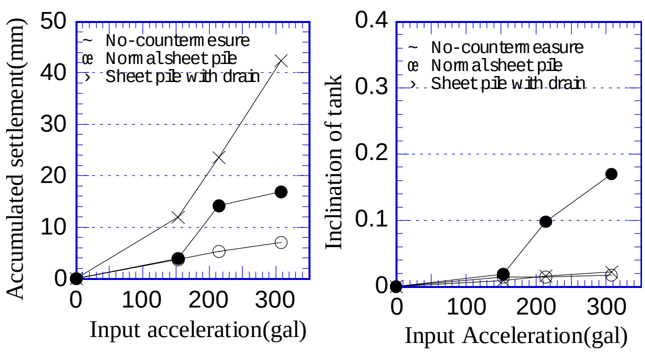
TEST RESULTS

Figure 6 shows the vertical distribution of the maximum excess pore water pressure ratio Ru at the acceleration level of 150gal. Figure 6 (a) and (b) are those under the tank and outside of the tank. It is observed that Ru under the tank without the countermeasure does not reach 1.0 under the influence of the initial shear stress and the large deformation of the foundation soil. On the contrary, Ru under the tank with the normal sheet pile enclosure approximately reaches 1.0 due to the confinement of the foundation soil under the tank. It is also possible that difference of initial stress condition influenced pore pressure increase, as the tank was set on the ground after the installation of the sheet pile ring. It is also recognized that the sheet pile with drain capability can prevent the foundation soil under the tank from liquefaction, although the effect of drain capability is not clear outside of sheet pule enclosure. This seems to be due to the difference of the boundary condition on drainage between the inside and the outside of the enclosure. Figure 7 shows the variation of the accumulated settlement and the accumulated inclination of the tank with input acceleration. Effectiveness of the normal sheet pile enclosure is confirmed as regards the reduction of the settlement, but the significant inclination is observed. This is due to liquefaction of the foundation soil under the tank. On the contrary, test results indicate that the sheet pile with drain capability is effective to reduce both the settlement and the inclination of the tank.



(a) under the tank (b) outside of the tank

Figure 6: Vertical distributions of maximum excess pore water pressure ratio



(a) accumulated settlement (b) inclination

Figure 7: Variation of accumulated settlement and inclination of the tank with input acceleration

COUNTERMEASURE FOR UNDERGROUND STRUCTURES

Underground structures of relatively light weight, such as common utility ducts, are often damaged due to the uplift displacement by liquefaction. This is due to the lateral flow of liquefied sand into the area below the structure where the overburden pressure is lower than in the vicinity. The authors have investigated the installation of steel sheet pile walls at the both sides of the structure as a countermeasure and the applicability of the sheet pile with drain capability to that method [Tanaka et al., 1996]. In this paper, results of shaking table tests are reported which have conducted to verify the effectiveness of the sheet pile with drain capability, considering the similitude [Iai, 1988] on permeability.

TEST PROCEDURE

Figure 8 shows the set up of the model tests and the locations of gages. The conditions for tests can be found in Table 3. The soil container used for tests is 2800mm in length, 845mm in depth and 695mm in width. The dimension of the model is determined on the basis of the similitude as that is an approximately one–ten geometrical scale model. The sand layer of silica sand with a mean diameter of 0.18mm and the uniformity coefficient of 1.4 was saturated with the cellulose solution to adjust permeability coefficient to about 7.5×10^{-4}cm/sec. The average unit weight of the sand layer in these tests was 1.90gf/cm^3. The underground structure model is a rigid box of 400mm wide, 250mm high and 570mm long and its apparent unit weight is 0.80gf/cm^3. Sheet pile models fixed at the bottom of the container are steel plate with dimensions of 800mm in height, 685mm in width and 3.0mm in thickness. Those with drain capability are equipped with many vertical drain pipes on the inner surface of the steel plate, where inner means the side enclosed by sheet piles. The flexural rigidity of drain pipes is negligible. Each model was excited by 30 cycles of sinusoidal motion, as shown in Fig. 9, at the maximum acceleration level of 100, 150 and 300gal. One level of input acceleration was applied during one test run and this was repeated for each acceleration level.

TEST RESULTS

Figure 10 shows the variation of the accumulated uplift displacement of the structure with input acceleration. Effectiveness of the sheet pile walls is recognized particularly for that with drain capability: no significant uplift is observed even during severe 300gal excitation. Figure 11 shows the horizontal distributions of excess pore water pressure ratio at the depth of 650mm, in which Fig (a) ,(b) are those for the cases of normal sheet pile and that with drain capability. It can be found in Fig.(b) that the sheet pile with drain capability was able to reduce excess pore water pressure just in limited area in their vicinity. Nevertheless, this seems to have played a significant role to prevent uplift displacement of the structure, confining the movement of the structure due to the

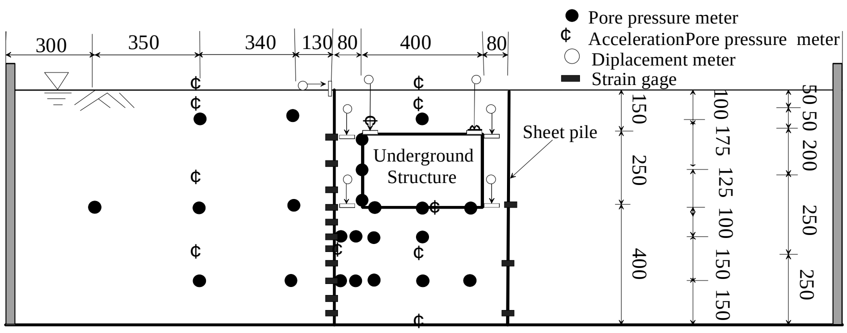


Figure 8: Model of shaking table tests on the countermeasure for the underground structure

Type of countermeasure	Dr of loose layer	condition of input motion
No-countermeasure	47.3%	100, 150, 300gal Sinusoidal motion shown in Fig.9 5Hz, 6sec
Enclosure with normal sheet pile	50.7%	
Enclosure with sheet pile with drain	55.1%	

Table 3: Conditions of shaking table tests

reduction of excess pore water pressure adjacent to the sheet pile wall, particularly between the structure and the sheet pile wall. Thus the sheet pile with drain capability is considered to be effective as the liquefaction countermeasure for underground structures.

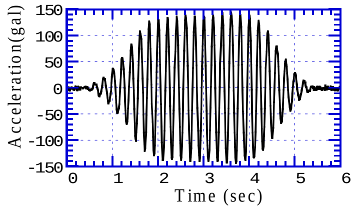


Figure 9: Example of input Acceleration

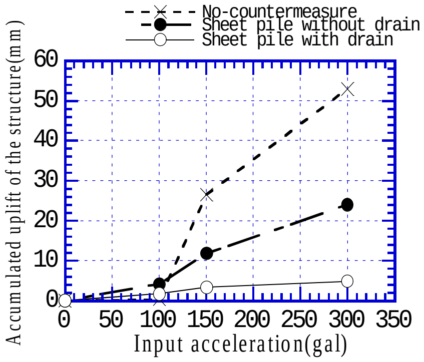
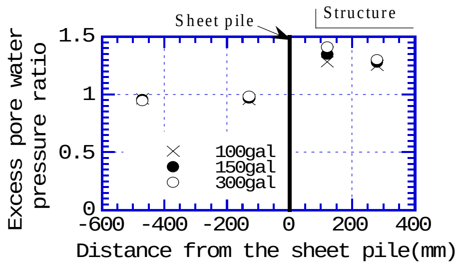
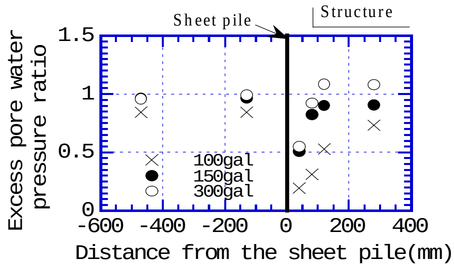


Figure 10: Relationship between accumulated uplift and input acceleration



(a) Sheet pile without drain



(b) Sheet pile with drain

Figure 11: Horizontal distribution of excess pore water pressure ratio

COUNTERMEASURE FOR PILE FOUNDATIONS AGAINST LATERAL FLOW OF LIQUEFIED

During 1995 hyogoken-nanbu earthquake, a lot of pile foundations were damaged by lateral flow of liquefied sand behind quay walls due to the horizontal movement, the inclination and the settlement of them. Thus, the authors conducted shaking table tests to propose a countermeasure with the steel sheet pile for pile foundations adjacent to quay walls [Yasuda et al., 1997]. Many tests were carried out parametrically under the various combinations of the location, the width and the flexural rigidity of the sheet pile wall.

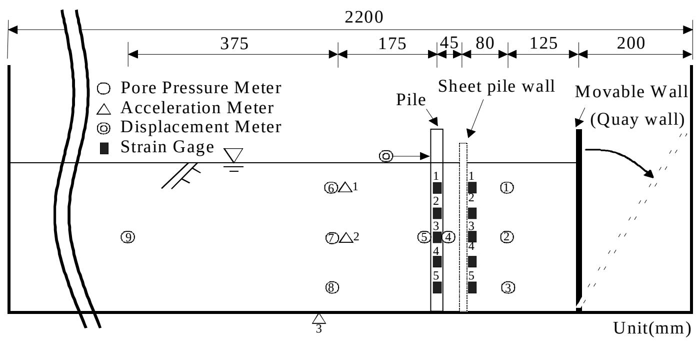


Figure 11: Model of shaking table tests

No.	Location of sheet pile wall	Width of sheet pile wall	Thickness of sheet pile wall
1	Upper side	150mm	1.2mm
2	Upper side	150mm	2.3mm
3	Upper side	150mm	3.2mm
4	Upper side	150mm	5.0mm
5	Lower side	150mm	1.2mm
6	Lower side	150mm	2.3mm
7	Lower side	150mm	3.2mm
8	Lower side	150mm	5.0mm
9	Upper side	100mm	2.3mm
10	Lower side	100mm	2.3mm
11	Upper side	50mm	2.3mm
12	Lower side	50mm	2.3mm

Table 4: Coditions of model tests

TEST PROCEDURE

Figure 11 shows the setup for model tests and the locations of gages. The conditions of tests can be found in Table 4. The soil container used for tests is 2200mm in length, 500mm in depth and 450mm in width. A model quay wall is placed at the location of 200mm from a side wall of the container. A pair of foam rubber sheets of 20mm in thickness are attached to the inner surfaces of the container and the model quay wall to generate uniform cyclic shear strain in the model ground during excitation. The model ground is a liquefiable sand layer prepared by pouring Toyoura sand with a mean diameter of 0.18mm from a certain height into water in the container. The pile foundation model of the acrylic plate with a thickness of 10mm and a width of 50mm is located at 250mm behind the quay wall model. The sheet pile wall models are steel piles of various dimensions of thickness and width shown in Table 1 and installed at the upper side or the lower side of the pile model, where lower means the side between the quay wall and the pile. Horizontal sinusoidal motion at a rate of 3Hz and at the acceleration level of 300gal was applied to liquefy the model ground and then, in 1 to 2 seconds after a stop of shaking, the model quay wall was moved quickly as shown in Fig.11.

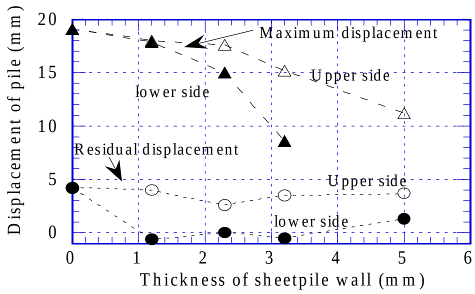


Figure12: Relationship between displacement of pile and thickness of sheet pile wall

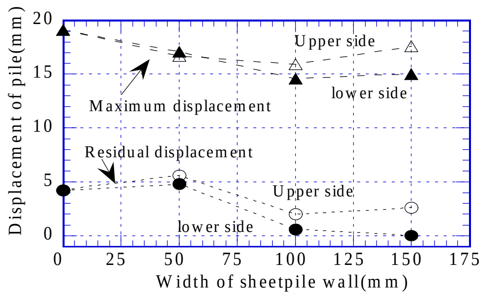


Figure 13: Relationship between displacement of pile and width of sheet pile wall

TEST RESULTS

Figure 12 shows the variation of induced displacement at the top of the pile with the thickness of the sheet pile wall. These are results from No.1 to No.8 tests using the sheet pile of 150mm width. In this figure, two kinds of displacement are pointed out which are the maximum displacement induced at the time of the quay wall movement and the residual displacement of the pile model. The test results indicated that the installation of the sheet pile wall at the lower side of the pile was effective and it could prevent the pile model from significant residual displacement. It is also recognized that the sheet pile wall with high rigidity was required to reduce the maximum displacement induced at the time of the quay wall movement. Figure 13 shows the relationship between the displacement induced at the top of the pile with the width of the sheet pile wall. These are results for the sheet pile wall of 2.3mm thickness. It can be seen that the effect to reduce the maximum and residual displacement are saturated at the sheet pile wall which is two times as wide as the pile model, whether the sheet pile wall is located at the lower or the upper side of the pile model. Thus, it is suggested from these test results that the sheet pile wall is effective which is installed at the lower side of the pile foundation and about two times as wide as the pile foundation.

CONCLUSION

Liquefaction countermeasures with the steel sheet pile wall for some kinds of structures were introduced and effectiveness of them was demonstrated through model tests and finite element analyses. The main results were as follows: (1) Results of finite element analyses indicated that the settlement of embankments due to liquefaction could be reduced by the installation of a pair of self-standing sheet pile walls at the toes of it and a certain degree of the effect could be expected even under the severe condition of liquefaction although effectiveness of that method tended to decline under that condition. (2) The effect of enclosing the foundation ground with the sheet pile wall as a countermeasure for tanks was confirmed in model test results, particularly for the steel sheet pile with drain capability: it could sufficiently prevent the tank from not only settlement but also inclination. (3)The steel sheet pile with drain capability was especially effective as a liquefaction countermeasure for underground structures. According to model test results, this resulted from the confinement of the structure due to the reduction of excess pore water pressure adjacent to the sheet pile wall, particularly between the structure and the sheet pile wall. (4)As a countermeasure for pile foundations against lateral flow of liquefied sand behind quay walls, the sheet pile wall was effective which was installed at the lower side of the pile foundation and about two times as wide as the pile foundation.

ACKNOWLEDGEMENT

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