Reduction in Liquefaction Induced Settlement of River Levee by Enhancing Horizontal Stress with Sheet Piles

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ABSTRACT

In order to reduce liquefaction damage to river levees, remedial stiff zones are constructed below toes of levees to restrain lateral flow deformation of liquefied foundation soil under the levee. Sheet piling, among other ground improving techniques, is preferably employed especially in urban areas where houses are existing close to levees and workspace is limited. Settlement of levee becomes smaller if sheet piles with a higher flexural stiffness are used. However, it is often the case that settlement does not satisfy the design criteria even if steel pipe sheet-piles with very high stiffness are used. In this study, an innovative countermeasure method using sheet piles is proposed. In the proposed method, the horizontal earth pressure is enhanced with the help of elastic nature of sheet piles. This is achieved by inserting a wedge along the sheet piles to the levee side which deforms laterally the soil below the levee, increases horizontal earth pressures and reduce the liquefaction susceptibility. The effectiveness of introducing additional horizontal stress, hereafter termed as pre-stress in this study, was examined through centrifuge tests. In the tests, the amount of the horizontal stress increase and the affected zone were confirmed by conducting cone penetration tests at multiple locations before and after application of the pre-stress. Dynamic centrifuge tests were carried out on models with and without the pre-stress. It was found that the cone tip resistance increased not only in the vicinity of the sheet pile but at almost everywhere in the soil beneath a levee. Settlement of the levee decreased with increasing the introduced pre-stress, confirming the effectiveness of the method.

Key words: *Sheet pile, River levee, Liquefaction countermeasure, Pre-stress*

1. Introduction

River levees in Japan have been repeatedly damaged by earthquakes and occasionally, hinterlands of the damaged levees were in danger of overflow of tsunami run up or flooding water due to the preceding heavy rain. Osaka city was the typical case just after Hyogoken Nambu earthquake in 1995, where a 6 m high Yodo River levee subsided more than 3 m due to the foundation soil liquefaction and river water almost reached the crest of the damaged levee. The Ministry of Construction has started remediation program to vulnerable levees. The major cause of damage to levees in earthquakes is soil liquefaction of the foundation soil or levee soil itself. Liquefied soil beneath levees may be squeezed vertically and flow laterally to the direction away from the levee centerline. Countermeasures in the remediation program, therefore, aim at mitigating crest settlements by providing containment for the deformation of the liquefiable foundation soils by forming remediated zones in liquefiable soil layer under embankment toes (Sasaki and Ishihara, 2016). Centrifuge tests (e.g. Okamura and Matsuo, 2002) and 1g shaking table tests (e.g. Ibi *et al.*, 2000; Sasaki *et al.*, 1992) have been conducted to explore effectiveness of remedial countermeasures beneath embankment toes for reducing crest settlement. In the above referenced studies, the remedial measures used in practice, including sheet pile enclosure, solidification and densification, were set beneath embankment toes or in the free fields just outside the toes. It has been shown that sheet piling is less effective for reducing embankment crest settlement as compared with other techniques. It is often the case especially in populated urban areas that available land area is not sufficient to execute ground improvement at around levee toe in the hinterland side and thus sheet piling is preferably employed. It is, therefore, needed to develop a novel sheet piling method to reduce crest settlement more effectively.

Fig. 1 shows a model of such centrifuge test before and after shaking events (Okamura and Matsuo, 2002). Sheet piles were installed vertically at both toes of the embankment with the tips sufficiently embedded to a non-liquefiable bearing layer. Subsidence of the embankment base line was attained its maximum under the crest and it became very small or even turned about upward heave in the vicinity of toes. The horizontal displacement constraint was imposed by the countermeasures, resulting in formation of a low strength exit path near the toes where the excess pore pressure ratio was high. The liquefied sand below the embankment was squeezed out from the embankment centerline towards free field exited through the path and heaved up the embankment toe. This observation suggests that prevention of heaving near the toe somehow will add further beneficial retrofitting effects.

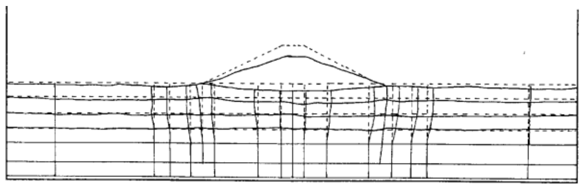

Fig. 1 Model with sheet pile enclosure at both toes after centrifuge shaking test (Okamura and Matsuo, 2002)

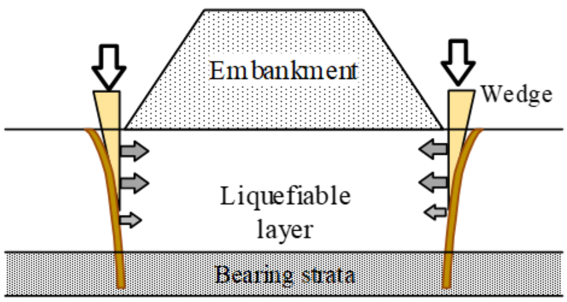

Fig. 2 Pre-stress introduced by wedge penetration

The authors have come up with an idea of introducing horizontal stress (hereafter in this paper, pre-stress) to a soil below the levee using sheet piles, in particular liquefiable soil below the embankment toes in the vicinity of sheet piles. The schematic illustration of the idea is shown in **Fig. 2**. Introducing the pre-stress to the soil can be achieved by inserting wedge along with the sheet piles in the embankment sides and the pre-stress will be maintained by the elastic nature of the sheet piles, which are installed into the stiff bearing layer through the liquefiable layers. The merits of the pre-stress in combination with sheet piles are; (1) the introduced horizontal forces to the soil below the embankment will be proportional to the horizontal deformation of pile head which is visibly identified, and (2) a creep deformation of soil may reduce horizontal stress of the soil with time but as far as the sheet piles are deflected, the pre-stress will keep acting.

As liquefaction resistance of a soil, that is cyclic stress ratio to liquefy the soil, is proportional to mean effective stress, the increase in the horizontal stress by $\Delta\sigma_x$ enhances the liquefaction resistance accordingly. For instance, for a soil with K_0 initially equals to 0.5 and after introducing pre-stress to 1.0 (ΔK_0= 0.5), the ratio of liquefaction resistance of soil from before to after introducing pre-stress is $(1+2K_0+\Delta K_0)/(1+2K_0)$= 1.25.

In this paper, a series of dynamic centrifuge tests on the effectiveness of pre-stress in reducing embankment settlement due to foundation liquefaction is reported.

2. Centrifuge test procedure

A series of centrifuge tests was carried out on model embankments resting on a liquefiable loose saturated sand with several different pre-stresses introduced by sheet piles.

Cone penetration tests were also conducted at locations in the models in order to better understand the extent of impact of the pre-stress introduced by the sheet piles on liquefaction resistance in the model.

2.1. In-flight CPT

A medium dense uniform dry sand layer was built as shown in **Fig. 3**. A sheet pile was set in a model container with its tip rigidly fixed to the container base. The head of the sheet pile was deformed horizontally to 5 mm or 10 mm and fixed with an electronic actuator. Dry Toyoura sand was rained to a relative density of 50% in the container with internal dimensions of 500 mm wide, 230 mm deep and 120 mm long. The model was set on a geotechnical centrifuge in Ehime University and brought up to 30g. A miniature cone penetrometer with a 6 mm diameter set on the two axes loading table was used to measure tip resistance profile. The cone was penetrated vertically at a constant rate of 6 mm/s at locations P1, P2 and P3. The fixation of sheet pile head was released which introduced increase in horizontal stress in the soil left hand side of the sheet pile. The cone penetration was successively conducted without stopping the centrifuge at another locations P4, P5, P6 and P7. The test conditions are shown in **Table 1**. In this study, two types of model sheet piles and two head deformations were tested. The model sheet piles were aluminum plates of 2mm or 3mm thick instrumented with strain gauges. The flexural rigidity of the model sheet piles scaled to that of a prototype sheet pile of 1.3 MPa m^4/m and 4.3 MPa m^4/m. The letter C and E refer to the CPT and the embankment

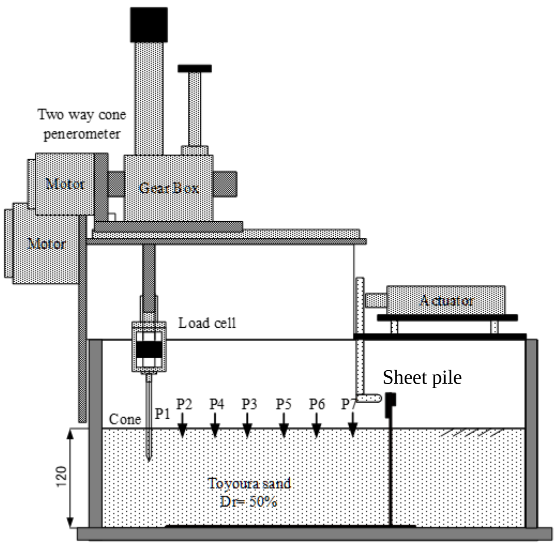

Fig. 3 Schematic illustration of model for CPT

test, respectively.

Table 1. Test conditions

	Thickness of model sheet pile	*Induced sheet pile head deflection*
	mm	mm
C0, E0	2.0	0
C1, E1	2.0	5
C2, E2	2.0	10
C3, E3	3.0	10

2.2. Shaking tests on foundation soil with embankment

Embank models shown in **Fig. 4** were prepared in the same rigid container. The procedure of building the foundation soil was the same as that for CPT tests. A set of sheet pile was fixed to the bottom of the container and the heads are horizontally displaced to 5 mm or 10 mm and locked with actuators before staring sand raining. During the sample preparation, accelerometers and pore pressure cells were installed at the proper locations and orientations. Potentiometers to measure settlement of embankment crest and base were also set after the embankment was set on the foundation soil.

The soil used to construct embankments was a mixture of Toyoura sand and Kaolin clay at 3:1 by dry weight. The soil was prepared at water content of 10% and compacted in a wooden mold of trapezoidal shape to a dry density of 90% the maximum dry density. The embankment in the mold was frozen in the refrigerator, set on the foundation layer in the container box and thawed in the room temperature. On completion of thawing the embankment, both the foundation soil and the embankment was fully saturated with viscous fluid in a vacuum chamber, to the degree of saturation higher than 99.5 %. The viscous fluid was prepared by dissolving 1.6% Metolose by weight in water, so as to achieve a viscosity of 30 times the viscosity of water (30cst kinematic viscosity).

The model was set on the centrifuge and the centrifugal acceleration was increased gradually to 30g. The viscous fluid on the ground and in the pore of the embankment soil was drained through a stand pipe connected to the bottom drainage holes until the ground water table stabilized at 10 mm below the ground surface. The actuators unlocked the displacement of the sheet piles head to apply pre-stress to the ground. It should be mentioned that the procedure to displace the sheet pile head at 1g and unlock at 30 g closely simulates application of pre-stress to the soil below the embankment in the field condition. However, the procedure reduced horizontal stress of the soil in the free field and this is not the case for the field condition. In this study, therefore, effects of pre-stressed sheet pile on the impact of mitigating soil liquefaction and embankment settlement are modelled conservatively.

The model was finally shaken horizontally with a mechanical shaker. The acceleration time histories of the event have the basic shape indicated in **Fig. 5**, with peak accelerations of approximately Ah = 1.6 m/s^2 and lasted 30 seconds in the prototype scale. Predominant frequency of the input motions was 0.7 Hz. The input motion in terms of shape of the waves and each peak acceleration were repeated quite well from test to test.

All the models tested in this study had the foundation soil depth of 120 mm (3.6m in prototype) and the embankment height of 50 mm (1.5m).

3. Results and discussion

All results, hereafter in this paper, are presented in prototype scale otherwise mentioned.

3.1. CPT

Fig. 6 depicts profiles of CPT tip resistance observed at several locations in the test C3. The tip resistance increased approximately linearly with depth. Tip resistance at P1 and P3 before unlocking the sheet pile head was lowest and increased as the distance from sheet pile decreased after unlocking the head.

It has been reported that cone tip resistance, qc, is sensitive to not only vertical stress but also horizontal stress and empirical relationships between qc and mean effective stress were proposed (e.g. Houlsby 1998, Jamiolkowski *et al*. 2003). Since liquefaction resistance of sand is proportional to mean effective stress (Ishihara, 1985), in-situ soil resistance to liquefaction is considered

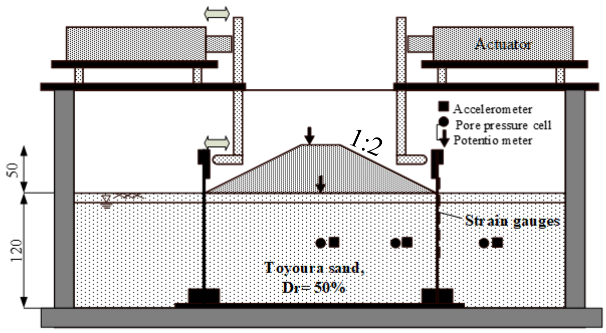

Fig. 4 Schematic illustration of model with embankment for shaking test

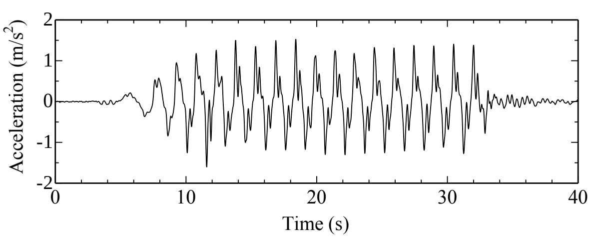

Fig. 5 Time history of input acceleration

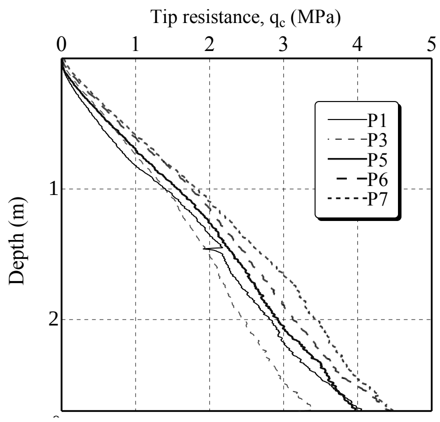

Fig. 6 CPT tip resistance observed for C3

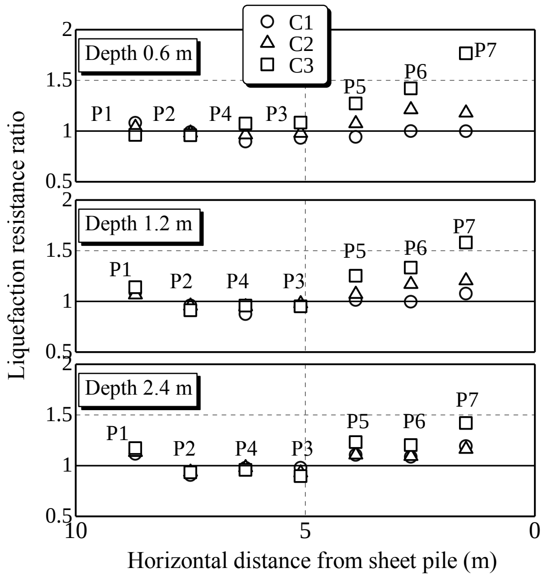

Fig. 7 Liquefaction resistance ratio

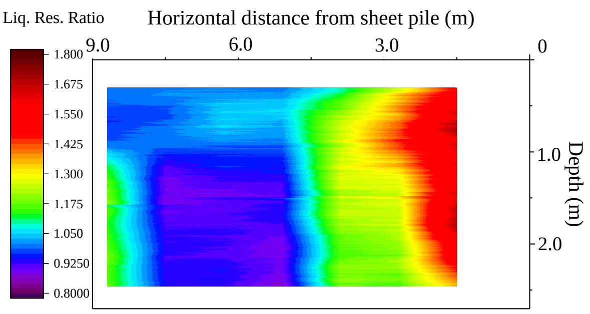

Fig. 8 Contours of liquefaction resistance ratio of C3

to be well estimated based on qc. Robertson and Weide (1998) proposed the following equation for clean sand;

$$\mathrm{CRR} = 93\left[\left(\frac{qc}{Pa}\right)\left(\frac{Pa}{\sigma\prime_{v0}}\right)^{n} \Big/ 1000\right]^{3} + 0.08 \qquad (1)$$

where CRR= cyclic resistance ratio to liquefaction, Pa= atmospheric pressure, σ_{v0}'= effective vertical stress and n= exponent that varies between 0.5 and 1.0 (Olsen, 1997). Distribution of liquefaction resistance are estimated with **Eq. (1)** with the exponent n= 1. The liquefaction resistance in the vertical axis of the **Fig. 7** are normalized with respect to that before introducing pre-stress, i.e. average at P1, P2 and P3. Liquefaction resistance ratio begins to increase at lateral distance from the sheet pile L= 4.5m. The ratio increased as L decreases, with the ratio being higher for shallower depth. For test C3 the liquefaction resistance of the soil near the sheet pile is believed to have enhanced approximately 1.8 times. Distribution of the ratio of C3 are also shown in **Fig. 8**. Note that the effect of the left boundary on CPT tip resistance at greater depth is reflected in the liquefaction resistance ratio. Therefore, the liquefaction resistance at P7 may also be influenced by the right boundary condition (i.e. the existence of the sheet pile) as well as by the horizontal confinement due to the elastic reaction of the sheet pile.

3.2. Dynamic tests of model with embankment

In the centrifuge at 30g, the fixed sheet piles head was released before the shaking events. The strain measured with strain gauges attached on the sheet pile for E1 and E2 are indicated in **Fig. 9**. The change in strain was due to releasing the force at the pile head with the actuators as well as change in earth pressures on the sheet pile. Change in earth pressure was estimated based on the observed strain distribution of the sheet piles and found that the coefficient of horizontal earth pressure at rest increased in the order of 0.2 and 0.4 for E1 and E2, respectively. Strain of the sheet pile was unfortunately not observed in E3.

Excess pore pressures during the shaking event for E0 and E3 are compared in **Fig. 10**. The soil at three locations of pore pressure cells seems to have liquefied during shaking and continued to liquefy after certain duration before the pressure started to dissipate. This is not only for test E0 without pre-stress but also for E3 with the largest pre-stress. Knowing the liquefaction resistance of medium dense Toyoura sand to be approximately 0.15, the factor of safety against liquefaction, FL, for the shaking acceleration of 1.6 m/s^2 is 0.47 for the level ground

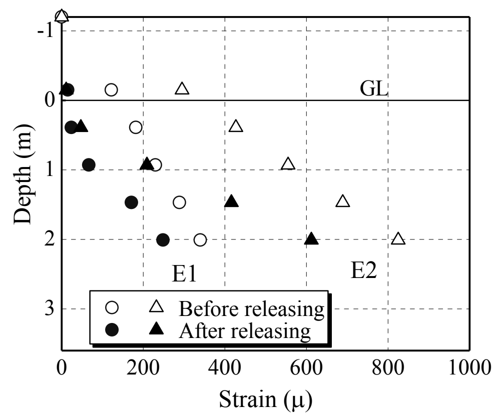

Fig. 9 Strain of sheet piles before and after releasing the fixing

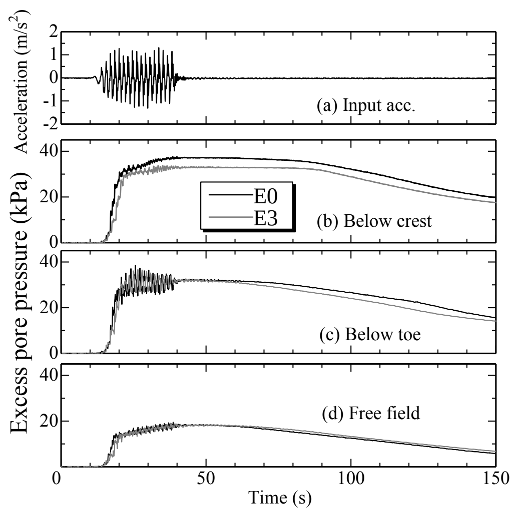

Fig. 10 Excess pore pressure responses for tests E0 and E3

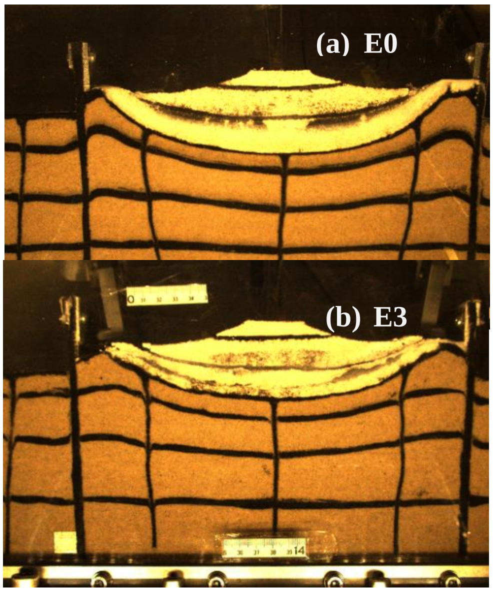

Fig. 11 Photos of model E0 and E3 after first shaking event

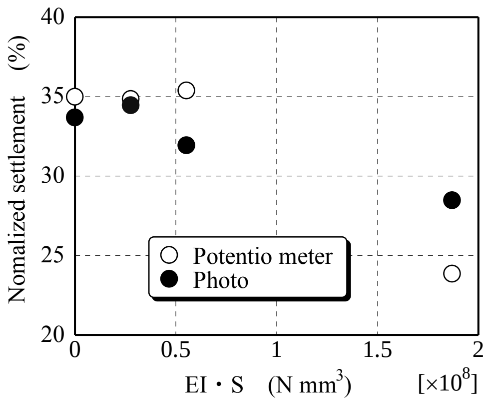

Fig. 12 Reduction in crest settlement with pre-stress

condition. Although the liquefaction resistance enhanced approximately 1.8 times in the vicinity of the sheet plie, FL value was 0.85, and thus still blow the unity. The consequences that all the models have liquefied is believed to be reasonable. Excess pore pressures below the crest and toe indicated in **Fig. 10(b)** and **(c)** for the E3 are somewhat lower and duration of liquefaction lasted after the shaking ceased was shorter as compared to those for E0. This difference is believed to be associated with the pre-stress introduced to the model E3.

Fig. 11 presents photos of E0 and E3 after shaking. Deformation of the foundation soil, especially in the zone near the sheet piles below embankment toes are reduced for the case of E3. Crest settlement normalized with regard to the initial embankment height are shown in **Fig. 12**. The abscissas, product of flexural rigidity of the sheet pile and the initial imposed deformation, indicates an index representing pre-stress. The crest settlement reduced in the model E3.

Since the sheet piles used in this study were rather flexible, the soil liquefied and significant settlement occurred even for the tests with pre-stress. But further reduction of crest settlement can be expected if sheet piles with higher rigidity are used.

4. Conclusions

In order to improve the effectiveness of sheet pile enclosure technique on mitigation of levee settlement, pre-stress sheet pile method was proposed. The effects of the method were verified with a series of centrifuge tests. It was confirmed that by introducing horizontal stress increment in the soil below embankments, extent of liquefaction and crest settlement were mitigated.

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