THE DYNAMIC BEHAVIOR OF A STEEL PIPE SHEET PILE FOUNDATION IN A LIQUEFIED LAYER DURING AN EARTHQUAKE

Nguyen Thanh TRUNG¹, Osamu KIYOMIYA² and Makoto YOSHIDA³

¹Member of JSCE, Doctoral Student, Dept. of Civil & Env. Eng., Waseda University
(Okubo 3-4-1, Shinjuku-ku, Tokyo 169-8555, Japan)
E-mail: nguyenthanchung@ruri.waseda.jp

²Member of JSCE, Professor, Dept. of Civil & Env. Eng., Waseda University
(Okubo 3-4-1, Shinjuku-ku, Tokyo 169-8555, Japan)
E-mail: k9036@waseda.jp

³Member of JSCE, Penta-Ocean Construction Co. Ltd.
(Nasushiobara, Shikuchou 1534-1 Tochigi 329-2746, Japan)
E-mail: Makoto.Yoshida@mail.penta-ocean.co.jp

Various forms of damage to the bridge foundation structure in the revetment along riverbanks and sea coasts caused by liquefaction had been observed during past earthquakes. Several studies on liquefaction using physical model tests and numerical analysis have been conducted in recent years. However, few studies have investigated the seismic behavior of the foundation in a revetment with a slope. In strong earthquakes, the sloped ground is expected to be unstable, and lateral spreading of the ground may occur simultaneously with the loss of soil strength in the liquefaction layer. Moreover, in the seismic design specification (JRA-2002) of the bridge, the liquefaction verification of the foundation is stipulated for a flat ground but not for a sloped ground. Therefore, the effect of the lateral pressure of the liquefaction layer on the foundation in the revetment must be investigated further. This study aims to investigate the dynamic behavior of a steel pipe sheet pile (SPSP) foundation of a cable-stayed bridge and its effect on the performance of the superstructure in the revetment with a slope. A 1-G shaking table test with a scale of 1:60 was conducted on a flat model and a slope model of 15°. In addition, 2-D numerical modeling was applied in an effective stress analysis method that was used on a multi-spring model and cocktail glass model. The differences in the dynamic responses between the two models clearly illustrate the significant effect of the ground slope on the seismic behavior of the SPSP foundation and superstructure.

Key Words: SPSP foundation, liquefaction, shaking table test, effective stress analysis

1. INTRODUCTION

Various forms of damage to the pile and caisson foundation structures had been observed in areas of liquefaction in past earthquakes^{1), 2)}. Some of these were pile failures near the bottom of the liquefied layer, whereas others were pile failures near the pile head. These failures were likely caused by the liquefaction that occurred due to a decrease in the soil strength and lateral movement of the liquefied layer. Moreover, significant damage was observed at both the pile body and pile head in the sites located near or on the revetment with a sloped surface ground along riverbanks or sea coasts^{3), 4), 5)}. This damage was likely caused by the unstable ground during liquefaction. In recent years, many important lessons and

insights regarding the basic mechanisms of soil-pile interaction in liquefied soil and their effect on the performance of a superstructure during liquefaction have been understood from field observations, shaking model tests, and numerical analysis. However, most of these studies were conducted on flat ground or ground with a mild slope line for a pile foundation structure. Ramin *et al.*⁶⁾ conducted a large shaking table test on the pile foundation near a gravity-type quay with flat ground. Haeri *et al.*⁷⁾ and Ramin *et al.*⁸⁾ investigated the response of a group of piles to liquefaction-induced lateral spreading by large-scale shake testing using a sloped ground with an angle of 5°. In addition, Tokida *et al.*⁹⁾ conducted tests on various sloped ground models at 5° angle and varying slope lengths. Miyajima *et al.*¹⁰⁾ performed a

Fig.1 Tower of Cable-stayed Bridge.

shaking table test and determined that the pile response depends on the sloping surface of the ground, ranging from 2° to 6° . Tokimatsu *et al.*^{11), 12)} studied the seismic behavior of soil-pile-superstructure system during soil liquefaction and liquefaction-induced ground displacement by shaking table test. Therefore, the researches simulated the dynamic behavior of pile foundation on the flat ground or with mild slope from 2° to 6° . However, the SPSP foundation, a type of caisson foundation, works not only as a support structure but also as a retaining wall in the revetment, that may not have been discussed before. Consequently, in this study, the behavior of SPSP foundation with a slope of 15° will be investigated.

Moreover, in the current bridge seismic design specification by JRA *et al.*¹³⁾, the verification of the liquefaction of the foundation structure is stipulated for flat ground. The verification of liquefaction-induced lateral spreading is conducted for a foundation that is less than 100 m from the water-front. The foundation in the revetment with a slope, whether affected by liquefaction-induced lateral spreading or not, is not clearly mentioned, thus further investigations and studies are required.

In this study, a 1-G shaking table test with a 1:60-scale model was designed for two test models of SPSP foundation to determine the behavior of the bridge foundation during vibration. The first model was on a flat ground surface (denoted by the flat model), and the second model was on a 15° -sloped ground (denoted by the slope model). Additionally, a 2-D numerical finite element method using the effective stress analysis (ESA) and eigen value analysis techniques was conducted to simulate the behavior of

the liquefied ground and bridge foundation during vibration. The eigen value analysis technique was used to investigate the dynamic characteristics of models. Furthermore, the ESA technique was used to consider the liquefaction of loose sand for both drained condition and undrained condition.

2. SHAKING TABLE TEST

This study was conducted using the 1-G shaking table facility of the Penta-Ocean Construction Corporation in Japan. The shaking table test was designed for both the sloped and flat models to investigate the difference in the dynamic response between the models under liquefaction conditions, as described in the following sections.

(1) Prototype

The tower and superstructure of a cable-stayed bridge supported by the SPSP foundation on the ground was modeled in the shaking table test. The outline of the tower is shown in Fig.1. The foundation has 165 steel pipe piles with dimensions of 36.456 m length and 29.469 m width. Each steel pipe pile has a diameter of 150 cm and a thickness of 2.5 cm.

To simplify the structure for constructing the physical model, the superstructure-tower system of the prototype was modeled as a single degree of freedom system. The mass of the system includes the mass of the superstructure and tower at the top of the column. The natural frequency of the system (f_L) was calculated as suggested by Yoneda¹⁴⁾ as follows:

Photo 1 1-G shaking table test on the flat model.

Table 1 Scaling factors of shaking model test.

Parameter	$\lambda = \text{prototype} / \text{model}$	Scale
Length	λ	60
Density	1	1
Time	$\lambda^{0.75}$	21.56
Stress	λ	60
Pore water pressure	λ	60
Displacement	$\lambda^{1.5}$	464.76
Acceleration	1	1
Strain	$\lambda^{0.5}$	7.75
Water permeability coefficient	$\lambda^{0.75}$	21.56
Bending stiffness	$\lambda^{4.5}$	100,387,728

$$f_L = \frac{1}{2\pi} \sqrt{\frac{\sum_0^i 3EI / (\alpha h)^3}{m_e}} \quad (1)$$

where f_L is natural frequency; m_e is the mass of the superstructure; h is the height from the bottom of the pier to the position of the lowest cable; α is the factor that depends on the ratio of stiffness between the tower and girder; and I is the area moment of inertia of the tower.

(2) Test model and material properties

All material properties of the physical model and ground were scaled using a similitude law suggested by Iai¹⁵⁾. **Table 1** summarizes the scaling factors applied in this shaking model test.

Photo 1 shows a set-up of the flat model on the shaking table. The test model includes the pier, superstructure, and foundation in the sand ground. The natural frequency of the pier-superstructure system of the model was determined by the dynamic characteristics of the single-degree-of-freedom system of the prototype using a scale of 1:60. The pier in the model consists of four steel columns that are rigidly fixed together by a steel plate at the top with a mass of 60 kg. Each column has dimensions of 1.1 m

Fig. 2 Grain size distribution of Yamagata sand.

height and a cross section in a tubular shape of 2.27 cm diameter and 0.19 cm thickness. The foundation is a caisson made of acrylic materials with a dimension of 49 cm width, 60.8 cm length, and 83.4 cm height. The cap at the top of the foundation is an acrylic plate that is 60.8 cm long, 49 cm wide, and 9.8 cm thick. The footing of the pier is constructed of steel with dimensions of 26.6 cm length, 46.6 cm width, and 18.5 cm thickness.

The ground in the models consists of a 48.8 cm liquefiable sand layer with a relative density of 50% using Yamagata sand No. 6 ($D_{50} = 0.3$ mm) overlying a 74.3 cm non-liquefiable layer with a relative density of 90%. The soil layers of the model ground were constructed using the sand drop method. The sand was gradually dropped into the vessel up to the water level step. However, the relative density of the non-liquefiable layer was controlled by the amount of tamping and the measured weight of the sand layer. The thickness of the sand layer for each tamping period was 10 cm.

The rubble layer consists of Grade 6 crushed stone with a particle size of 13–20 mm. The grain size distribution of the Yamagata sand used for the ground is provided in **Fig. 2**. The slope of the ground in the case of the slope model was 15° in the longitudinal direction.

The initial shear modulus (G_0) of the sand layer was calculated using the following equation:

$$G_0 = \rho(v_s)^2 \quad (2)$$

where G_0 is the initial shear modulus, ρ is the mass density of sand, and v_s is the shear wave velocity of the soil layer.

The pulse method was used to determine the shear velocity of the sand.

An impulsive sin wave (amplitude: 100 Gal, period: 0.0176 s) was inputted at the bottom of the shaking table by the electrohydraulic vibration machine. The acceleration responses at two locations, one at the top of the soil layer and another at the bottom of the layer, were recorded in time-history waves to capture the difference in the peak time between the two locations. Then, the shear wave velocity (v_s) was calculated by the following equation:

$$v_s = \frac{H}{\Delta T} \quad (3)$$

where v_s is the shear wave velocity, H is the height of the soil layer or distance between the two locations, and ΔT is the difference in the peak time between the two locations.

(3) Instrument and deployment

The instruments and their placements are shown in **Figs.3** and **4**, respectively. The accelerometers and pore water pressure transducers were arranged in the near- and far-field areas of the ground at various depths of the liquefaction and non-liquefaction layers. The accelerometers were attached at the top and bottom of the pier. Two horizontal laser displacement transducers were installed at the top and bottom of the pier, and two vertical displacement transducers were installed at the bottom of the pier. The strain gauges were installed on opposite sides of the foundation at different depths. The small, circular targets were embedded in the ground surface to record their movements before and after shaking.

(4) Base excitation

The models were shaken with a base harmonic acceleration at a constant frequency of 10 Hz. The duration time was 2 s. The amplitude increased incrementally from 50 to 300 Gal, and one of the input stages is shown in **Fig.5**.

The frequency and wave numbers of input ground motion was selected in the consideration of the subduction zone earthquakes (level 2 earthquake motion) and the similarity law.

Fig.5 Acceleration wave input of 50 Gal at the base.

3. NUMERICAL ANALYSIS

The 2-D finite element method model was adopted to simulate the behavior of foundation and soil-structure interaction. First, the eigen value analysis was used to determine the dynamic characteristics of the two models before liquefaction in this analysis. Eigen value analysis was conducted by the TDAP III program (developed by Taisei Corporation, Tokyo, Japan). Second, the ESA technique was used both by the multi-spring and cocktail glass models suggested by Iai¹⁶⁾. The FLIP program (developed by Port and Airport Research, Institute, Yokosuka, Japan) was used in this analysis.

(1) Calculation model

The boundary at the bottom of the model was fixed in the vertical and horizontal directions, and the lateral boundary at the two sides was fixed in the horizontal direction. The walls of the SPSP foundation, and the acrylic plate at the top of the pier were modeled as elastic beam elements. The steel footing plate, acrylic cap of the piles, and partition walls for longitudinal bridge axis were modeled as plane strain elements. The soil was modeled as plane strain elements. Along the soil-wall interface, the walls and adjacent soil elements were connected by a few springs in the vertical and horizontal directions. The horizontal springs were modeled as cut-off tension springs. The thickness of the plane strain element was the same as the width of the test vessel. By modeling the pile in the 2-D calculation model, nonlinear spring elements between the pile and ground are usually adopted to consider soil movement among piles: the 3-D effect. By modeling the steel pipe sheet pile foundation, side wall friction is considered by the spring element. This spring property is the same as that of the vertical spring at the front wall.

(2) Effective stress analysis (ESA)

In the ESA, the nonlinear dynamic analysis was conducted by the time-history direct integration method. The boundary condition of pore water in ESA using the cocktail glass model was considered in the undrained conditions (no seepage) at side walls and bottom wall of the test vessel. The numerical

Fig.6 Numerical slope model in Effective Stress Analysis (ESA).

Fig.7 Joint property of interface element between soil and the foundation.

Fig.8 Stress-strain curves from liquefaction parameters.

Fig.9 Multi-spring model in effective stress analysis.

Fig.10 Cocktail glass model in effective stress analysis.

model in the analysis is shown in **Fig.6**.

Fig.7 shows the property of the joint element between soil and foundation on two sides (front and back side of the foundation) and at the bottom of foundation. The effect of side friction of the foundation is not considered because this friction force is small during liquefaction of the ambient loose sand.

The influence of stress (or strain) history on cyclic deformation-strength characteristics of soil in the liquefaction layer is shown in **Fig.8**. The figure shows the relationship between stress and strain of liquefaction layer in both the indoor three-axial vibration test and ESA.

The soil was modeled as plane strain elements

Table 2 List of soil parameters.

	Parameter	Symbol	Liquefaction layer	Non-liquefaction layer	Rubble layer
Parameters for deformation characteristics	Wet unit weight	ρ (t/m ³)	1.96	2.05	1.37
	Initial shear modulus	G_{ma} (kPa)	3,866	21,788	2,993
	Initial bulk modulus	K_{ma} (kPa)	10,083	56,819	7,805
	Standard confining pressure	σ_{ma}' (kPa)	2.27	6.85	0.28
	Poisson's ratio	ν	0.33	0.33	0.33
	Internal friction angle	ϕ_f (degree)	36.55	42.80	41.60
	Hysteretic damping ratio	h_{max}	0.24	0.24	0.24
Parameters for Muti-Spring model	Phase transformation angle	ϕ_b (degree)	28	-	-
	Overall cumulative dilatancy	w_1	8.2	-	-
	Initial phase of cumulative dilatancy	p_1	0.45	-	-
	Final phase of cumulative dilatancy	p_2	1.07	-	-
	Threshold limit for dilatancy	c_1	4.48	-	-
	Ultimate limit of dilatancy	S_1	0.005	-	-
Parameters for Cocktail glass model	Reduction factor of bulk modulus for liquefaction analysis	r_K	0.5	-	-
	Power index of bulk modulus for liquefaction analysis	l_K	2	-	-
	Parameter controlling dilative and contractive components	r_{ed}	0.5	-	-
	Parameter controlling contractive component	r_{ed}^c	2	-	-
	Parameter controlling initial phase of contractive component	q_1	1	-	-
	Parameter controlling final phase of contractive component	q_2	1	-	-
	Limit of contractive component	ε_d^{cm}	0.5	-	-
	Small positive number to avoid zero confining pressure	S_1	0.005	-	-
	Parameter controlling elastic range for contractive component	c_1	4.48	-	-

using two liquefaction models of the loose sand.

The first model is called a multi-spring model and is shown in **Fig.9**. The multi-spring model is a strain-space multiple-mechanism model. This model considers the effect of the rotation of the principal stress axes on the cyclic behavior of the sand. The effect results in the rise of pore water pressure under the undrained conditions. As shown in this figure, pore water pressure increases by calculation steps. The second model is a cocktail glass model improved from a strain-space multiple-mechanism model in the drained condition suggested by Iai *et al.*¹⁷⁾, as shown in **Fig.10**. Migration analysis was carried out at every calculation steps and pore water pressure decreased.

There are two main assumptions in this model. First, the volumetric strain is decomposed in a dilative component and contractive component, as determined in Eq.(4). The dilative component affects the dissipation of pore water pressure in the steady state and the horizontal displacement response. The second is a relationship between relative velocity and coefficient of permeability determined in Eq.(6). This assumption influences the rate of pore water

development and dissipation as follows:

$$\varepsilon_d = \varepsilon_d^c + \varepsilon_d^d \quad (4)$$

where ε_d is the volumetric strain; ε_d^c is the contractive component; and ε_d^d is the dilative component.

Ozutsumi¹⁸⁾ presented the migration of water obtained by the multi-dimension equation of consolidation by Biot:

$$\{k(\theta)h_i\}_i - \left(\frac{\partial \delta_{i,i}}{\partial t} \right) S_r - C(\psi) \left(\frac{\partial h}{\partial t} \right) = 0 \quad (5)$$

where $k(\theta)$ is the coefficient of permeability; ψ is the pressure head; $C(\psi) = n \left(\frac{\partial S_r}{\partial \psi} \right)$ is the relative water content; n is porosity; S_r is the degree of saturation; h is hydraulic gradient; and δ is displacement. k is determined by the sand size and the void ratio in the test vessel.

The coefficient of permeability (k) suggested by Chapuis and Aubertin¹⁹⁾ for sand used in the cocktail

Fig.11 Fundamental frequencies of the flat and slope models in initial state.

Fig.12 Time history of EPWP ratio at W4 and W8 in the flat model under 300 Gal.

Fig.13 Time history of EPWP ratio at W4 and W8 in the slope model under 300 Gal.

glass model as follows:

$$k = C \frac{g}{\mu_w \rho_w} \frac{e^3}{S^2 D_R^2 (1 + e)} \quad (6)$$

where k is the coefficient of permeability; C is a constant; μ_w is the dynamic viscosity of water; ρ_w is the density of water; D_R is the specific weight of sand; S is the specific surface; and e is the void ratio.

The liquefaction parameters of the soil layers for the multi-spring and cocktail glass models were determined using the shear modulus and relative density of the sand, among other parameters, which are summarized in **Table 2**. The hydrodynamic pressure acting along the slope surface of the revetment was considered using fluid elements. The numerical integration was performed using the Wilson- θ method with $\theta = 1.4$. A Rayleigh damping method with parameters $\alpha = 0$ and $\beta = 0.002$ was used to ensure the numerical stability of the analysis. In the analysis, the self-weight analysis step was conducted first to calculate the initial stress and strain of the model before the calculation of the dynamic analysis.

Table 3 Result of the eigen value analysis.

Mode No	Flat model		Slope model	
	Frequency (Hz)	Mode damping (%)	Frequency (Hz)	Mode damping (%)
1	5.97	3.21	5.86	3.21
2	16.84	9.63	16.56	9.56

4. RESULTS AND COMPARISONS

(1) Eigen value result

Table 3 summarizes the fundamental frequencies and mode damping of both models when liquefaction does not occur. Their modal mode of the first natural frequency is presented in **Fig.11**. This table illustrates that both the first and second frequencies in the slope model were lower than those in the flat model. However, the difference between the frequencies in the two models was quite small. The mode damping of the slope model was almost identical to that of the flat model. Therefore, there was a slight difference in the dynamic characteristics of the two models in the initial state when liquefaction did not occur.

Fig.14 EPWP ratio distribution in the flat model under 300 Gal in the ESA multi-spring.

Fig.15 EPWP ratio distribution in the slope model under 300 Gal in the ESA multi-spring.

Fig.16 EPWP ratio in the flat model from 50 to 300 Gal.

Fig.17 EPWP ratio in the slope model from 50 to 300 Gal.

Fig.18 Comparison of EPWP ratio at W4 and W8 between the flat and the slope models.

(2) Comparison of the ground motion between the effective stress analysis and the experiment

A comparison was conducted between the experiment and the ESA using the multi-spring model (ESA-multi-spring) and the cocktail glass model (ESA cocktail glass).

a) Excess pore water pressure of the surface layer

The time histories of the EPWP at points W4 in the near field and W8 in the far field of the experiment and ESA under the 300 Gal input ground motion are shown in **Figs.12** and **13** for the flat and slope models, respectively. The EPWP ratio did not exceed 0.8 and perfect liquefaction did not occur at 300 Gal. The results of the EPWP among the experiment, multi-spring model, and cocktail glass model are fairly different from each other. The liquefaction start time was almost identical for both methods, whereas the EPWP dissipation, maximum EPWP ratio, and vibration components differed considerably. The EPWP ratio gradually decreased after the vibration stopped in the experiment; this phenomenon can be explained using the cocktail glass model. The EPWP ratio was almost the same between the experiment and the cocktail model after 12 s. However, the generation and dissipation of EPWP in the model occurs very quickly during vibration time of 2 s. The cocktail glass model displayed a vibration component of the EPWP. We assumed that the quick dissipation of EPWP was due to the large value of coefficient of permeability of the test sand and low value of water viscosity. These items are limitations

Fig.19 Time history of acceleration at AH8, AH10 and AH19 in the flat model under 300 Gal.

Fig.20 Time history of acceleration at AH7, AH9 and AH16 in the slope model under 300 Gal.

Fig.21 Comparison of acceleration at AH13, 16 in the flat model and AH16, 19 in the slope model.

of the 1-G vibration model test. We also assumed that the vibration component was due to unstable calculation of double integrations of the constitutive equation.

The EPWP ratio distribution of the ground under 300 Gal in the ESA multi-spring is shown in **Figs.14** and **15** for the flat and the slope models, respectively. The EPWP ratio reached approximately 1.0 at the surface liquefied layer in both models after approximately 4 s. However, the EPWP was not uniform at the surface layer in the multi-spring model.

The EPWP ratios from the 50-300 Gal input ground motions are shown in **Figs.16** and **17** for the flat and the slope models, respectively. The EPWP

ratio increased due to the increase in input ground motion in the multi-spring model. In the vibration, test, the EPWP ratio increased to 0.7 even though the input ground motion was 100 Gal. The EPWP ratio at W4 given by the calculation was approximately 0.2. The sand of liquefied layer in the vessel may have been looser than expected at this point.

Fig.18 compares the EPWP at W4 and W8 between the slope and flat models obtained by the experiment and ESA from 50 to 300 Gal. There are six red points in **Fig.18**, and their values gradually increase. The EPWP ratio of the slope model at W4 was 1.1÷1.25 times higher than that of the flat model in both the experiment and ESA. At W8 the EPWP ratios of ESA using both the multi-spring and cocktail models were nearly identical between the two models, but in the experiment, the ratios of the flat model were larger than those of the slope model.

b) Acceleration of the surface layer

Fig.19 presents the time histories of the horizontal accelerations at points AH8, AH10, and AH19 in the flat model. In the non-liquefaction layer, the acceleration at AH8 of ESA corresponded well with that of the experiment, and the acceleration did not exhibit any amplitude variations during the shaking period. Meanwhile, the acceleration at the near-field AH10 and far-field AH19 of the liquefaction layer varied significantly starting at 7.5 s, and this amplitude gradually decreased between 7.5 and 10 s, as shown in **Fig.19**.

The horizontal accelerations at points AH7, AH9, and AH16 in the slope model are shown in **Fig.20**. These points are at the same position, corresponding

Fig.22 Measured displacement distribution of the ground surface movement in the slope model under 300 Gal.

Fig.23 Measured displacement distribution of the ground surface movement in the flat model under 300 Gal.

to points AH8, AH10, and AH19 in the flat model. Similar to the acceleration behavior in the flat model, the acceleration amplitude in the liquefaction layer at points AH9 and AH16 decreased and did not appear in the non-liquefaction layer at AH7.

However, **Fig.20** illustrates that the amplitude of the acceleration toward the water at AH9 and AH16 became larger compared to that of the acceleration toward the land in the vibration test. The instability of the slope ground generated the high acceleration in the direction of the water. However, the calculation did not provide the same result as the test. **Fig.21** presents a comparison of the horizontal accelerations at AH13, AH16 in the slope model and at AH16,

AH19 in the flat model. The difference in the acceleration in the near field between AH13 in the slope model and AH16 in the flat model was minimal in the 50-150 Gal cases in both the experiment and ESA. However, the acceleration ratio steadily approached 2:1 in the 150-300 Gal cases. The acceleration at AH13 in the flat model became approximately 1.5 times that of the slope model in the experiment under 300 Gal. In the far field, the acceleration at AH19 in the flat model was less than that at AH16 in the slope model, approximately 1.5 times in both the experiment and ESA. The ESA using multi-spring and cocktail glass models had the same trend in the acceleration development in the experiment.

Fig. 24 Measured residual displacement at T1-c, T3-6, T3-11, T5-c and the footing in the slope model from 50 to 300 Gal.

Fig.26 Vertical displacement on ground in the slope and flat models under 300 Gal.

c) Displacement of the surface ground

Fig.22 displays the distribution of the measured residual horizontal displacement at the maximum input ground motion of 300 Gal in the slope model. The figure illustrates that the movement of the slope ground was in the direction from T3-11 toward the land to T3-6 toward the water when liquefaction occurred. The displacement distribution from T1-6 to T5-6 in front of the foundation was nearly uniform. However, the displacement at T3-6 in front of the foundation was slightly smaller than that of the other points. The displacement of the points near the foundation was less than that in the free field from T1-11 to T5-11 behind the foundation. The residual displacements at the front and back of the foundation were smaller than those of other points. A lateral relative soil movement around the foundation occurred due to sand liquefaction in the slope model. The foundation that was inserted into the non-liquefaction layer blocked the horizontal displacement of the points in the free field.

The measured horizontal displacement value of the points near the foundation is shown in **Fig.24**. The figure illustrates that in the 50-100 Gal cases, there was a slight difference in the displacement between points, including points T3-6, T3-11, T1-c, and T5-c near the front, back, and two sides of the foundation, respectively. However, the difference became significantly larger in the 150-300 Gal cases. The displacement of the footing was the smallest in

Fig. 25 Measured residual displacement at T1-c, T3-6, T3-11, T5-c and the footing in the flat model from 50 to 300 Gal.

these cases. The foundation clearly retained the movement of soil on the back side and pushed the soil forward on the front side.

Fig.23 presents the distribution of the measured residual horizontal displacement under the 300 Gal case in the flat model. The figure illustrates that the displacement distribution on the back side of the foundation from T1-6 to T5-6 was nearly uniform. However, the displacements of points from T2-11 to T4-11 on the front of the foundation were larger than those of T1-11 and T5-11 in the free field. Moreover, the displacement on the right side was larger than that on the left side. The foundation vibration created a disturbed adjacent sand layer.

Fig.25 presents the horizontal residual displacement values of points near the foundation in the 50-300 Gal cases for the flat model. The figure illustrates that from 50 to 150 Gal, there was a slight difference in the displacement among points, including points T3-6, T3-11, T1-c, and T5-c of the front, back, and two sides of the foundation, respectively. The movement of the ground was from T3-6 to T3-11 to the right side of the foundation. The movement was in the opposite direction from 150 to 300 Gal.

Fig.26 presents the vertical displacement of the free field near the foundation at T3-12. The displacement of the multi-spring model was approximately four times less than that of the cocktail glass models. The ESA using the cocktail glass model corresponded more closely with the results of the experiment than did the ESA using the multi-spring model with regard to the vertical displacement.

d) Acceleration of the superstructure

Fig.27 presents the time histories of the horizontal acceleration of the superstructure and pile cap of the flat model under the 300 Gal input ground motion in both the experiment and the ESA.

For the superstructure, the acceleration at AH4 of the experiment was nearly identical to that of the ESA in the multi-spring and was larger than that of the cocktail model by approximately 1.5 times. The acceleration of the experiment was larger for the pile

Fig.27 Time history of horizontal acceleration response of superstructure and pile cap in the flat model under 300 Gal.

Fig.29 Maximum horizontal acceleration of the superstructure AH4 and pile cap AH3 in the flat model from 50 to 300 Gal.

Fig.31 Comparison of acceleration at AH3 and AH4 between the flat and slope models.

cap at AH3, with a difference between the values of 9%-12%. Additionally, **Fig.29** illustrates that the acceleration development of AH4 and AH3 from 50 to 300 Gal in the vibration test was in good agreement with that of the ESA.

Fig.28 presents the time histories of the horizontal acceleration of the superstructure and pile cap for the

Fig.28 Time history of horizontal acceleration response of superstructure and pile cap in the slope model under 300 Gal.

Fig.30 Maximum horizontal acceleration of the superstructure at AH4 and pile cap at AH3 in slope model from 50 to 300 Gal.

300 Gal input ground motion in the slope model. The accelerations of the superstructure and pile cap of the experiment were larger than those of the ESA from 9% to 21%. **Fig.30** illustrates that the accelerations of the superstructure and pile cap display a similar trend in both the experiment and the ESA. The difference in the maximum acceleration amplitude between the multi-spring and cocktail models was quite little.

Fig.31 presents a comparison of the maximum horizontal accelerations of the superstructure and pile cap in the experiment and ESA under the 50-300 Gal input ground motion in both models. The accelerations are nearly identical between the flat and slope models. The difference in acceleration was minimal in the 50-100 Gal cases but a little larger in the 100-300 Gal cases.

e) Displacement of the superstructure

Fig.32 presents the time histories of the horizontal and vertical displacements of the pile cap and superstructure in the flat model in both the experiment and the ESA. The displacements at DH1 and DH2 in the experiment were larger than those in the ESA. The displacements in the ESA cocktail glass model were considerably smaller than the displacements in other cases.

Fig.32 Horizontal and vertical displacement response of superstructure and pile cap in the flat model under 300 Gal.

Fig.33 Horizontal and vertical response of superstructure and pile cap in the slope model under 300 Gal.

Fig.34 Maximum horizontal displacement of the superstructure DH2 and pile cap DH1 in the flat model from 50 to 300 Gal.

Fig.35 Residual horizontal displacement of the superstructure DH2 and pile cap DH1 in the flat model from 50 to 300 Gal.

Fig.36 Maximum horizontal displacement of the superstructure DH2 and pile cap DH1 in slope model from 50 to 300 Gal.

Fig.37 Residual horizontal displacement of the superstructure DH2 and pile cap DH1 in the slope model from 50 to 300 Gal.

Fig.38 Comparison of maximum horizontal displacement at DH1 and DH2 between the flat and slope models.

A residual displacement was observed in the experiment. However, a small residual displacement was calculated by the ESA. The vertical displacement at DV1 and DV2 in the 300 Gal case in the experiment was approximately 1.5 times less than that in the ESA multi-spring and the displacements in the ESA cocktail glass were smallest. The maximum and residual displacements during shaking from 50 to 300 Gal are shown in **Figs.34** and **35**, respectively. There was a remarkable agreement between the experiment and ESA multi-spring for the maximum and residual displacement between 50 to 150 Gals; however, the displacements in the ESA multi-spring were considerably less than those in the experiment for the 150-300 Gal cases. Based on JRA-2002, the allowable displacement of 4.9 mm for the top of the foundation was calculated by multiplying the width of the foundation by 1%. Thus, when liquefaction occurred, the maximum horizontal displacements of the pile cap under the 300 Gal input ground motion was approximately 0.35 times less than the allowable displacement for the experiment, and 0.2 times less than that for the ESA multi-spring. Moreover, the maximum displacements in the ESA cocktail glass were approximately 2 times less than those in the experiment. The residual displacements were very small and not significant during shaking.

Fig.33 presents the time histories of the horizontal and vertical displacements at the pile cap and superstructure in the slope model under 300 Gal. The residual displacement calculated in the ESA and the difference in the maximum displacement between the ESA and experiment were rather large. The displacement of DV1 exhibited a downward trend and that of DV2 exhibited an upward trend. This result of vertical displacement indicates that the foundation rotated and inclined toward the left. The maximum and residual displacements during shaking from 50 to

Fig.39 Maximum vertical displacement of the pile cap at DV1 and DV2 in the flat model from 50 to 300 Gal.

300 Gal in the slope model are shown in **Figs.36** and **37**, respectively. Compared with the allowable displacement of the pile cap, the maximum displacement at the pile cap was approximately 0.7 times less than that in the experiment, 0.4 times less than that in the ESA multi-spring, and 0.15 times for ESA cocktail-glass. The maximum horizontal displacement at the pile cap satisfied the allowable design value. Moreover, from 50 to 150 Gal there was a good agreement in the maximum and residual displacement between the experiment and ESA multi-spring; however, in the 300 Gal case, the displacements in both the ESA multi-spring and cocktail glass was much less than that in the experiment.

Fig.38 illustrates that when the range of the maximum displacement is 0-1 cm in the 50-100 Gal cases, the difference in the displacement of the superstructure and pile cap between the flat and the slope models was minimal, and their ratio was approximately 1:1. The range of the displacement increased for the 100-300 Gal cases, and the difference progressively increased; the displacement ratio approached 1:2, indicating that the displacement of the slope model became approximately twice that of the flat model in this experiment. The ESA had the same trend with the vibration test. However, the differences in displacement between the two models were smaller. **Fig.39** presents the maximum vertical displacements in the flat model during shaking under the 50-300 Gal input ground motion. There was a remarkable agreement between the experiment and ESA multi-spring. Moreover, there was a slight difference in the displacement value between DV1 and DV2 in both the experiment and ESA multi-spring. The settlement of the foundation in the flat model was almost even. The displacements in the ESA cocktail glass were much smaller than those of other cases.

Fig.40 Maximum vertical displacement of the pile cap at DV1 and DV2 in the slope model from 50 to 300 Gal.

Fig.41 Comparison of the inclination of foundation between the flat and slope models from 50 to 300 Gal.

Fig.42 Relation of displacement time history between DH2 and DH1 in the flat model of experiment.

Fig.43 Relation of displacement time history between DH2 and DH1 in the slope model of experiment.

Fig.44 Relation between acceleration at AH3 and displacement at DH1 time histories in the slope model of experiment.

Fig.40 presents the maximum vertical displacements in the slope model during shaking under the 50-300 Gal input ground motion. There was a slight difference in the displacement value between DV1 and DV2 from the 50 to 200 Gal input ground motion in both the experiment and ESA multi-spring. However, the difference between the values became larger, and the displacement at DV2 was approximately three times larger than that at DV1 for both the experiment and ESA multi-spring in the 300 Gal case. The displacements in the ESA cocktail glass were also much smaller than other cases. Based on Eq.(7), the inclination in this case was 0.23% in the experiment. The inclined settlement of the founda-

tion model occurred in the slope. The inclination of the foundation was determined by the following equation:

$$\alpha = \frac{DV2 - DV1}{L} 100 \quad (7)$$

where α is the inclination of the foundation (%); $DV1$ and $DV2$ are the residual values of vertical displacements at the top of the footing (mm); and L is the distance between $DV1$ and $DV2$ (266 mm).

Figs 39 and 40 show almost constant displacements from 50 to 150 Gal. This is because in the lower input ground motion, the inclination of foundation was small, thus the maximum vertical displacement increased a little. However, in the higher input motion, the inclination became larger, and the residual displacement component increased more significantly. The difference in maximum vertical displacement was also more significant.

Fig.41 presents the inclination of the foundation in the experiment and ESA under the 50-300 Gal input ground motion in both models. In both the experiment and ESA multi-spring, the difference in the inclination between the two models was minimal for the 50-150 Gal cases. However, in the 300 Gal case, the inclination in the slope model was approximately 0.2 and 0.05 times less than the allowable inclination (1%) for the experiment and ESA multi-spring, re-

Fig.45 Bending and axial strain in experiment in the flat model.

Fig.46 Bending and axial strain in experiment in the slope model.

Fig.47 Bending and axial strain in ESA in the flat model.

Fig.48 Bending and axial strain in ESA in the slope model.

spectively. While, in the flat model, the inclination was approximately 0.11 times less than the allowable value for the experiment and 0.03 times less for the ESA multi-spring. The inclinations in ESA cocktail glass were very small in both models.

The comparison of horizontal displacement time histories between DH1 and DH2 in the flat and slope models in the vibration test are shown in **Figs.42** and **43**. It shows that in the case of 100 Gal, most of displacements at DH1 were much less than and out of phase with those at DH2. However, when the input ground motion increased to 200 and 300 Gal, the soil

layer was liquefied more significantly and frequency of the layer also reduced significantly; the hysteresis loop extended widely to the area where the superstructure and pile cap almost moved in the same horizontal direction during liquefaction. Their difference became smaller. These were in agreement with the results by Tokimatsu *et al.*¹¹⁾. In this vibration test, higher second and third vibrational modes were dominant in 100 Gal and the first vibrational mode was dominant during liquefaction

Fig.44 shows the relationship between acceleration at AH3 and displacement at DH1 in the slope

model. Amplitude of the acceleration wave form did not decrease even after liquefaction. The shape of the loop was almost constant during vibration.

f) Behavior of the foundation

The maximum bending and axial strain distribution in the experiment along the foundation in both the flat and slope models from 50 to 300 Gal are shown in **Figs.45** and **46**, respectively. The bending strains dominated the axial strain when the input acceleration amplitude was less than 100 Gal. The distribution of the axial and bending strains was uniform along the foundation depth. When liquefaction occurred, the strain increased at the bottom of the foundation rather than at the upper location.

The strains of the ESA multi-spring are shown in **Figs.47** and **48**. Generally, **Figs.45**, **46**, **47**, and **48** illustrate that the bending strains in both the experiment and ESA multi-spring reached a maximum value near the bottom of the pile foundation at S2 in both models.

Fig.49 Comparison of maximum strain along the front and back sides between the flat and slope models.

Fig.50 Maximum shear strain distribution in the flat model under 300 Gal.

Fig.51 Maximum shear strain distribution in the slope model under 300 Gal.

Fig.52 The ground reaction stress distribution of the SPSP foundation in the slope model.

Fig.49 presents a comparison of the maximum bending strain and axial strain of the pile foundation between the two models from 50 to 300 Gal in both the experiment and the ESA.

The maximum bending strain of the flat model in the experiment was almost larger than that of the slope model in the 50-150 Gal cases. However, when the liquefaction process was complete, the strain of the slope model became 1.5 times larger than that of the flat model in the 300 Gal case. For the ESA using both multi-spring and cocktail models from 50 to 200 Gal, the difference in the bending strain between the two models was minimal, and in the case of 300 Gal, the strain of the slope model was approximately 1.3 times larger than that of the flat model. The result of the experiment also illustrates that the maximum axial strain in the slope model was approximately 1.5 times larger than that of the flat model. However, the axial strain difference in the ESA between the two models was small.

Moreover, ESA multi-spring had the same trend as that of the experiment; however, the strain in the slope model was slightly larger than that in the flat model.

g) Strain of the ground and reaction stress

Fig.50 presents the maximum shear strain distribution of the surface layer using the ESA multi-spring. Large strain values were calculated in the area around the foundation in both the liquefaction and non-liquefaction layers. The strains were also quite large in the far field. Moreover, the maximum strains in the 0.008-0.01 range in the liquefaction layer distribution are presented by the black dashed line shown in **Fig.50**; the distribution had a symmetric pattern.

Fig.51 presents the maximum shear strains in the area around the foundation and on the down- and up-slope areas. The maximum strain distribution in the 0.04-0.07 range in the liquefaction layer is presented by the black dashed line in **Fig.51**; however, the distribution had an asymmetric pattern. The strain of the soil elements on the front of the foundation was larger than that on the back of the foundation.

Fig.52 presents the distribution of the reaction stresses along the foundation obtained by the ESA multi-spring in the slope model. The reaction stress became small in the liquefied layer but large in the unliquefied layer after liquefaction in both the flat and the slope models.

For the slope model, the reaction stress was small at both the front wall and the back wall. Inversely, the reaction stress at the back wall was large even though the sand was liquefied for the slope model. This reaction stress at the back wall pushed the foundation to move forward and rotated around the tip of the foundation.

The reaction stress of the bottom was nearly zero on the back side of the foundation, except for the area at the rear of the bottom due to the rotation of the foundation and cut off tension in the 300 Gal case. The foundation resisted the movement due to the unliquefied sand.

5. CONCLUSIONS

A vibration test and numerical analysis on a steel pipe sheet pile foundation were conducted on both slope and flat models to investigate their dynamic behavior on the SPSP foundation. The following conclusions can be drawn based on the results:

1) The horizontal response movement of the SPSP foundation increased due to the increase in the input acceleration in the flat and slope models in the vibration test. The lateral movement on the foundation became large when liquefaction occurred, and residual displacement at the top of foundation was observed for both models. The residual displacement in the slope model was considerably larger than that in the flat model.

2) In the slope model, the foundation moved down the slope and inclined in the shaking table test. The movement of the slope at the foundation that was inserted into the non-liquefaction layer was smaller than that of the free field.

3) The bending and axial strains along the foundation axial were nearly uniform before the liquefaction of sand occurred. When liquefaction occurred, the strains in the non-liquefaction layer became larger compared to the strains in the liquefied layer. The reaction stress of the slope model was small in the liquefied layer. The reaction force at the front wall was small in the liquefied layer for the slope model. However, the reaction at the back wall was large enough to move the foundation to the front direction. The foundation resisted the movement due to the non-liquefaction layer.

4) The effective stress analysis (ESA) has almost the same trend as the dynamic responses in the experiment. The difference in dynamic response of the foundation, superstructure, and ground between the flat and the slope models is minimal in the low-amplitude input ground motion, indicating that the effect of the ground slope is not significant. In cases of higher amplitude when liquefaction is observed, the effect of the ground slope becomes more significant, with the following trends: the slope causes an increase in the maximum and residual displacements of the pile cap and superstructure and a decrease in the horizontal acceleration. Furthermore, the slope causes an increase in the inclination of the foundation and the maximum value of the

bending and axial strain in the foundation pile. Therefore, the lateral movement of liquefaction layer due to the slope may partially affect the foundation during liquefaction.

5) The ESA using the multi-spring model can explain the behavior of the foundation with regard to maximum displacements, EPWP ratios, and bending strains during liquefaction. However, the calculated values of the residual displacement, etc. did not display a good agreement with the values observed in the vibration test.

6) The cocktail glass model that considers the dilative component of the sand and seepage of water can be used to estimate the dissipation of the pore water pressure and vertical displacement. However, the response displacement using the cocktail glass model is smaller than that using the multi-spring model. The cocktail model could explain the dissipation of the pore water pressure in the vibration test; however, the calculation result had the vibration component and was not stable. Methods for determining the parameters in the ESA using both the multi-spring model and the cocktail glass model to correlate the test results should be examined in future studies.

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