Reinforcement of River Embankment against the Nankai Trough Earthquake

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ABSTRACT

In the Great East Japan Earthquake of 2011, many river embankments were damaged by strong seismic waves. After the earthquake, tsunami spilled over damaged embankments. In the river levee, back covering coatings were damaged due to settlement of fill by the earthquake motions or liquefaction, and back covering coatings and fill soils flowed out backrush. As a reinforcement method for river banks, a construction method of placing sheet piles in the embankment wall body is being studied for the purpose of maintaining the function of the levee at the time of an earthquake or flood. The occurrence of the Nankai Trough earthquake is expected in Kochi Prefecture, and the prefecture is proceeding with countermeasures under the idea of triple protection. Especially reinforcement of river embankments is an important issue in disaster prevention and reduction. In the past, with respect to various reinforcement methods in river embankment, dynamic centrifugal model experiments and behavior by effective stress method are being verified. In this study, we conducted a model experiment of double wall sheet piles on shoulder of cut-off wall method. In addition, model experiments were also conducted on a floating structure that keeps embedded depth of sheet piles in the liquefaction layer and shorten embedded depth for the purpose of cost rationalization of the present construction method. Furthermore, the relationship with the dominant period of the earthquake was also investigated for these experiments.

Key words: *liquefaction, seismic response analysis, dynamic centrifugal force model experiment*

1. Introduction

On March 11, 2011, the Tohoku district Pacific offshore earthquake of magnitude 9.0 occurred. Due to this earthquake, in Tohoku district, large-scale embankment damages occurred simultaneously in many places, embankments did not work. Particularly, the damage from the tsunami increased around the Abukuma River and Natori River (Sendai River National Highway Office, 2017). In addition, most of the damage to the embankments were caused by liquefaction, and many embankments subsided by liquefaction.

In Kochi prefecture, the Nankai Trough earthquake of magnitude 9.0 class may also occur. The Nankai Trough Earthquake is expected to occur at a probability of 70% within the next 30 years (Kochi Prefecture, 2017). In order to minimize the damage from the Nankai Trough Earthquake, it is necessary to take countermeasures for liquefaction.

In this research, we will investigate the liquefaction countermeasure work against the earthquake of Level 1 (which occurs at a frequency of about once every several decades to several hundred years and causes great damage). We also investigate the predominant period of the seismic waves, which depends on the embedded depth of the sheet piles.

For the research method, effective stress analysis method LIQCA and dynamic centrifuge were used. We compared the results of the experiment and the analysis. We also confirmed the liquefaction behavior and examined the effectiveness of countermeasures.

2. Research case

In this study, three models were considered: Embankment model (**Fig. 1**), a model in which sheet piles were inserted to the mid-depth of the liquefaction layer (**Fig. 2**), and a model in which sheet piles were inserted to the base layer (**Fig. 3**). They are called **model 1, model 2**, and **model 3**, respectively. Model 2 is a case where no load is transmitted from the sheet pile to the base layer (called floating foundation). The dimensions shown in Fig. 1 are prototype size. Silica sand No. 5 with Dr = 90% was used for the base layer. Toyoura sand with Dr = 50% was used for the liquefaction layer. Silica sand No. 7 with Dr = 95% was used for embankment.

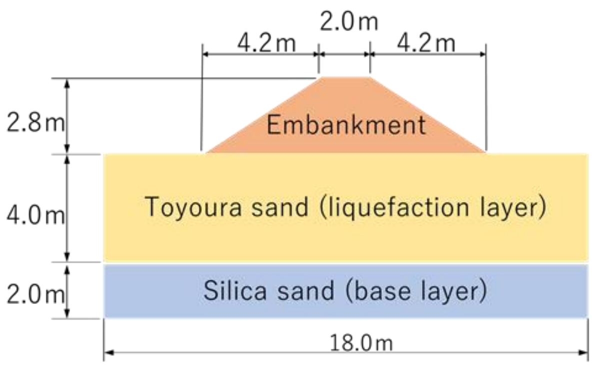

Fig. 1 Model1: Embankment ground model

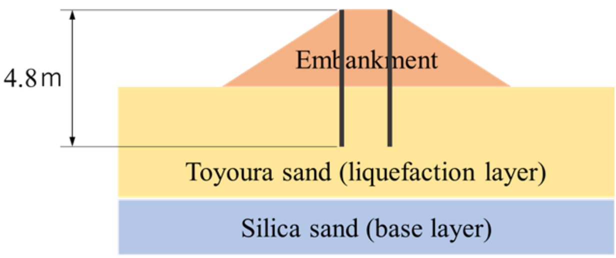

Fig. 2 Model2: Sheet piles are inserted to center of the liquefaction layer

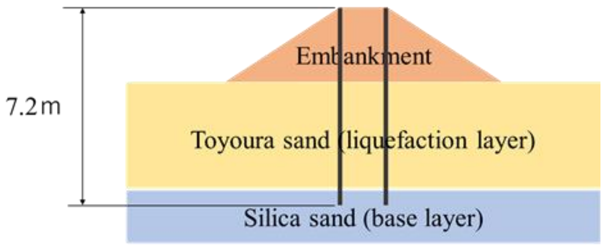

Fig. 3 Model3: Sheet piles are embedded to the base layer

2.1. Input seismic wave

For the seismic waves, the acceleration required to liquefy the experimental soil layer was calculated based on the simplified judgment method of the road bridge specification (Japan Road Association, 2012).

Fig. 4 shows the seismic waves used in the analysis. In the centrifuge model test, from the similarity rule, the acceleration was set to 80m/s^2 (40times) and the period was set to 0.067 sec (1/40).

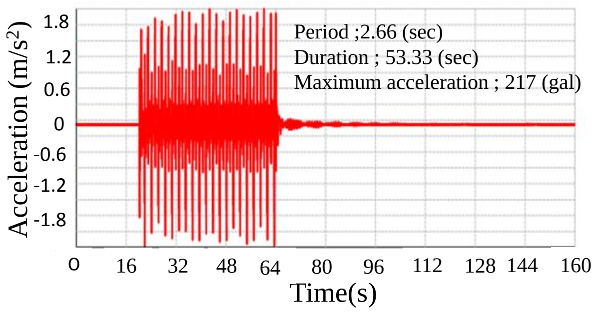

Fig. 4 Input seismic wave (prototype scale)

3. Liquefaction analysis

3.1. Analysis method

The analysis model was prepared with AUTOCAD and analyzed using the effective stress analysis method program LIQCA.

3.2. Analysis model

Fig. 5 shows the half section of the analytical model. For the analytical model, the width of the analytical model has been made sufficiently wide so that there is no influence of the propagation of the seismic wave on the lateral boundary. The drainage boundary was designated as the surface of Toyoura sand.

3.3. Material parameters

In this research, material parameters were set with

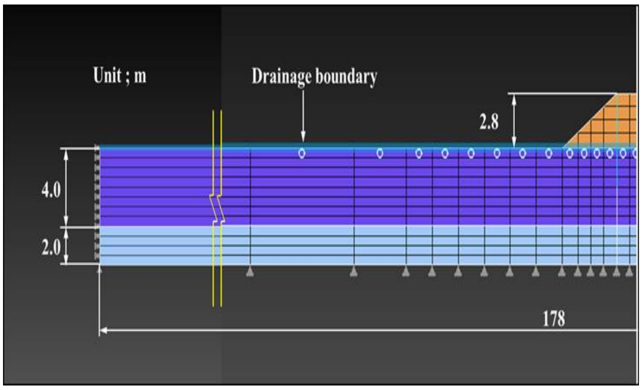

Fig. 5 Analysis model

reference to the simulation analysis of the centrifugal model experiments by Saito (2018). The parameters are shown in **Table 1** and **Table 2**.

Table 3 shows the physical properties of the sheet pile wall. As the material constant of the sheet pile, the value of the aluminum plate used in the centrifugal force model experiment was used.

Table 3. properties of the sheet pile wall

	Analysis Experiment	E (KN/m^2)	A (m^2)	I (m^4)	ρ (g/cm^3)
Sheet pile wall	Model 2	7.03×10^7	0.19	0.37	2.69
	Model 3		0.29	1.24	

Table 1. Parameters for Dynamic Analysis

	Embankment	Silica sand	Toyoura sand(Dr = 50%)	Toyoura sand(Dr = 70%)
Wet unit volume weight (γ t)	15.758(kN/m^3)	16.17(kN/m^3)	14.484(kN/m^3)	15.06(kN/m^3)
Saturated unit volume weight (γ sat)	18.7(kN/m^3)	19.845(kN/m^3)	18.774(kN/m^3)	18.8(kN/m^3)
Effective soil covering pressure	0(kN/m^2)	0(kN/m^2)	0(kN/m^2)	0(kN/m^2)
Static earth pressure coefficient	0	0	0	0
Nondimensional initial shear coefficient	873	1043	910	1040.9
Initial gap ratio (e_0)	0.856	0.6	0.791	0.718
Compression exponent (λ)	0.018	0.025	0.0039	0.0039
Swelling index (k)	0.006	2.0×10^{-4}	2.2×10^{-4}	2.2×10^{-4}
Pseudo-consolidation ratio	1	1	1	1.5
Dilatancy factor (D_0)	5	1	0.5	0.75
Dilatancy factor (n)	1.5	9	5	7
Water permeability coefficient / unit volume of water	8.67×10^{-5}(m/sec/kN/m^3)	1.0×10^{-3}(m/sec/kN/m^3)	1.0×10^{-4}(m/sec/kN/m^3)	1.0×10^{-4}(m/sec/kN/m^3)
Bulk elastic modulus of water	2000000(kN/m^2)	2000000(kN/m^2)	2000000(kN/m^2)	2000000(kN/m^2)
Transformation stress ratio (Mm)	0.909	0.909	0.909	0.817
Fracture stress ratio (Mf)	1.122	1.551	1.229	1.245
Parameter (B_0) in hardening function	2200	5000	3500	5185.7
Parameter (B_1) in hardening function	30	60	60	100
Parameter (Cf) in hardening function	0	0	0	0
Anisotropic loss parameter (Cd)	2000	2000	2000	2000
Plasticity reference strain (γP * r)	0.005	0.01	0.003	0.005
Elastic reference strain (γE * r)	0.01	0.2	0.006	0.02

Table 2. Parameters for Static Analysis

	Embankment	Silica sand	Toyoura sand (Dr = 50%)	Toyoura sand (Dr = 70%)
Wet unit volume weight (γ t)	15.758(kN/m^3)	16.17(kN/m^3)	14.484(kN/m^3)	15.06(kN/m^3)
Saturated unit volume weight (γ sat)	18.7(kN/m^3)	19.845(kN/m^3)	18.774(kN/m^3)	18.8(kN/m^3)
Poisson's ratio	0.333	0.333	0.333	0.333
Effective soil covering pressure	1.0×10^{-8}(kN/m^2)	1.0×10^{-8}(kN/m^2)	1.0×10^{-8}(kN/m^2)	1.0×10^{-8}(kN/m^2)
Static earth pressure coefficient	1	1	1	1
Proportionality coefficient of Young's modulus (E_0)	3494.07	1210.6	2775.2	2775.2
Constant (n)	1	1	1	1
Adhesive force	0(kN/m^2)	0(kN/m^2)	0(kN/m^2)	0(kN/m^2)
Internal friction angle	31.3(deg)	42(deg)	37.75(deg)	37.75(deg)

3.4. Analysis result

Fig. 6 and **Fig. 7** show the excess pore water pressure ratio distribution map after the earthquake. The blue color shows the excess pore water pressure ratio of 0, and the red color shows the excess pore water pressure ratio of 1.0.

As shown in **Fig. 6**, complete liquefaction did not occur under embankment. However, due to the rise of excess pore water pressure, the deformation of the embankment top, the slope and the horizontal ground near the embankment becomes larger.

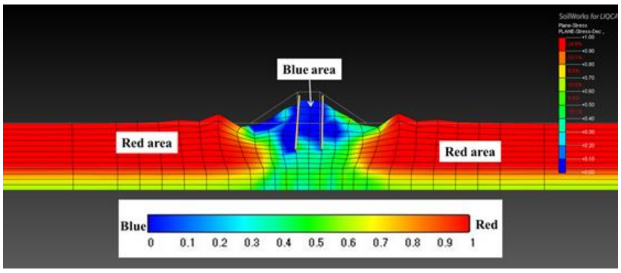

Fig. 6 Analysis results of model 2

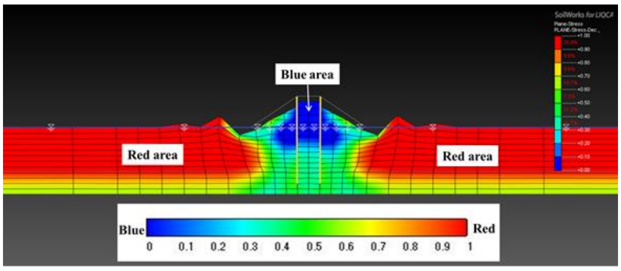

Fig. 7 Analysis results of model 3

As shown in **Fig. 7**, although the deformation was slightly smaller in Model 3, the behaviors of liquefaction were almost the same for each model.

4. Liquefaction experiment by dynamic centrifuge

4.1. Conditions of experiment

Liquefaction experiments were carried out using a centrifuge of Kochi National College of Technology. **Photo 1** shows the dynamic centrifuge .

The experiments were carried out in a centrifugal force field of 40 g. As shown in **Fig. 8**, the model size was set to a scale of 1/40 of the actual size (**Figs. 1 to 3**).

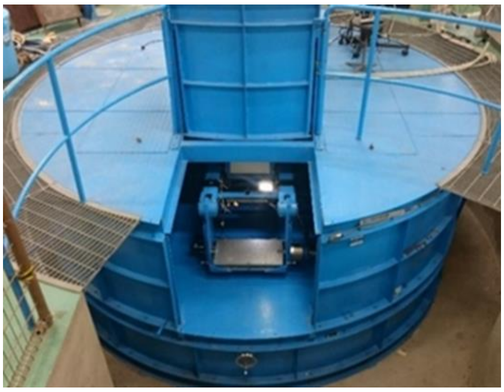

Photo 1. Dynamic centrifuge of Kochi National College of Technology

Seismic accelerometers and piezometers were installed at 2 cm, 6 cm and 10 cm depth from the ground surface.

Table 4 shows the ground condition of the centrifuge experiment.

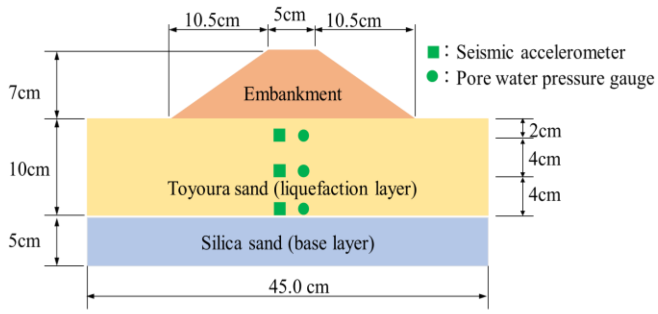

Fig. 8 Experimental model

Table 4. Grand condition of Centrifuge Experiment

	Wet unit weight (kN/m³)	Saturated unit weight (kN/m³)
Embankment (Silica Sand No. 7)	15.76	18.7
Liquefaction layer (Toyoura sand) Dr=50%	14.48	18.77
Basement layer (Silica Sand No. 5)	16.17	19.85

4.2. Experimental method

Photo 2 shows a photograph before the experiment of a model in which the sheet pile is inserted to the mid-depth of the liquefaction layer (model 2).

As mentioned earlier, the foundation layer was made of silica sand No. 5, and the liquefaction layer was made with Toyoura sand. The base layer and the liquefaction layer were saturated with a methyl cellulose solution having a viscosity 40 times that of water.

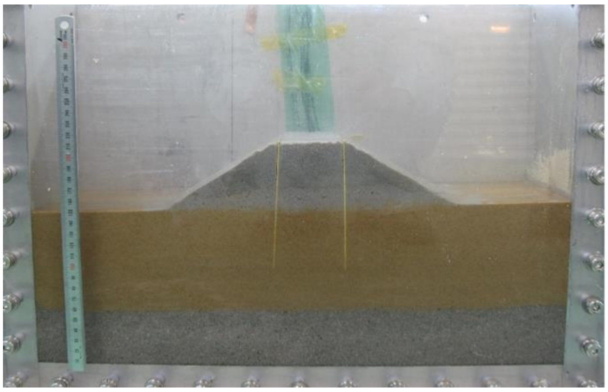

Photo 2. Soil layer of model 2 before experiment

Silica sand No. 7 was used as the filling material. The embankment was made by freezing with a hard aluminum form, thawed and placed on the liquefaction layer before the experiment.

4.3. Experimental result

Photo 3 shows a photograph after the experiment of model 2. Both of the embankment, and the liquefaction layer (Toyoura sand) subsided 3 mm. As the embankment settled the water level rose about 7 mm.

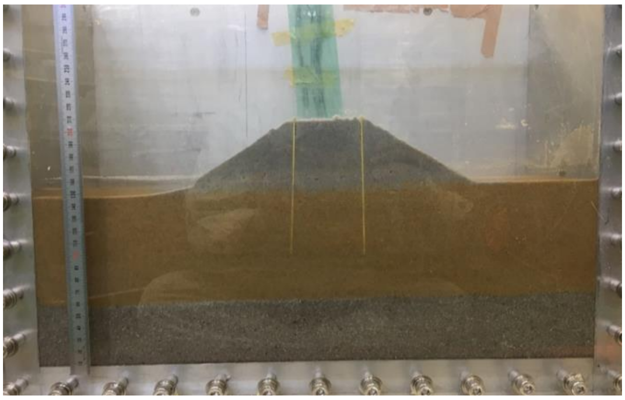

Photo 3. Soil layer of model 2 after experiment

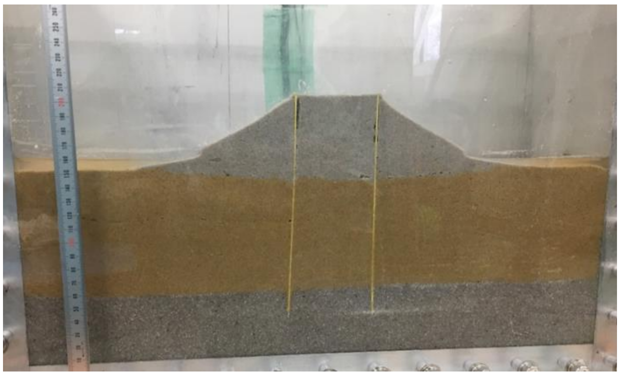

Photo 4. Soil layer of model 3 after experiment

Photo 4 shows a post-experimental picture of model 3 with sheet piles inserted to the base layer. The embankment settled about 2 mm. The liquefaction layer has slightly settled.

4.3.1. Excess pore water pressure

Fig. 9 shows the change of excess pore water pressure at the installation depth of 10 cm of piezometer for model 2. **Fig. 10** shows a similar figure for model 3. The effective stress is 65.9 kPa for both models.

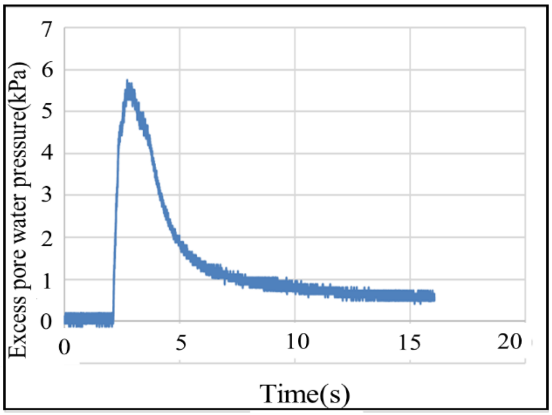

Fig. 9 Excess pore water pressure (10 cm) of model 2 (model scale)

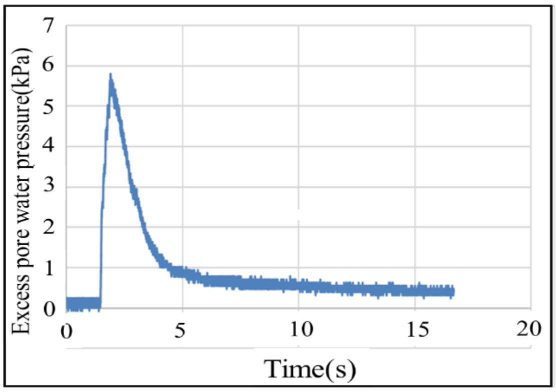

Fig. 10 Excess pore water pressure (10 cm) of model 3 (model scale)

The effective stress is 65.9 kPa for both models. In the model 2, pore water pressure u_w (35.4 kPa) + excess pore water pressure = 41.1 kPa.

In the model 3, pore water pressure u_w (38.2 kPa) + excess pore water pressure = 44.0 kPa.

Both models have not yet been completely liquefied. Moreover, it can be seen that excessive pore water pressure is increased by preventing pore water pressure dissipation in model 3 in which the sheet piles are inserted to the base layer.

4.3.2. Dominant period

Fig. 11 shows the earthquake Fourier spectrum of the model 2, and **Fig. 12** shows the earthquake Fourier spectrum of the model 3. As shown in **Fig. 11**, in model 2, the velocity response spectrum exceeded at a frequency of 10.5 Hz. The predominant period is 0.095 s, and on the prototype scale it is 40 times and it is 3.85 seconds. Likewise, as shown in **Fig. 12**, the model 3 has a predominant period of about 1.81 seconds.

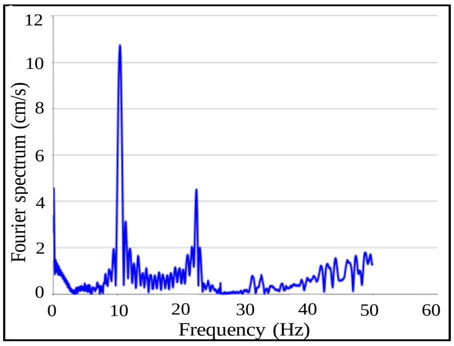

Fig. 11 Earthquake Fourier spectrum of model 2 (model scale)

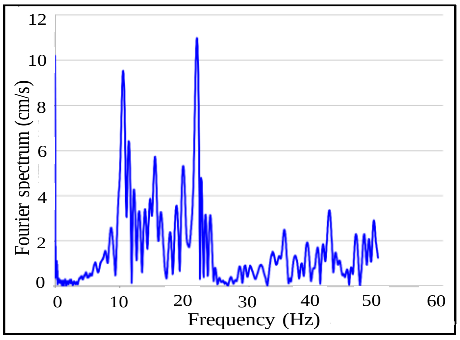

Fig. 12 Earthquake Fourier spectrum of model 3 (model scale)

That is, in the model 2, the predominant period has doubled. This indicates that resonance phenomena can be avoided by changing the predominant period according to the type of embankment and the natural period of the structure on the embankment.

5. Comparison of analysis result and experiment result

5.1. Excess pore water pressure ratio

In the analysis results of model 2 and model 3, the excess pore water pressure ratio just under the embankment did not reach 1.0, and perfect liquefaction did not occur. Also in the experimental results, the excess pore water pressure ratio did not reach 1.0 and perfect liquefaction did not occur. From this, it is considered that consistency between the analysis result and the experiment result is sufficient for the excess pore water pressure ratio.

5.2. Deformation

Table 5 shows the comparison of settlement. In the experiments, deformation of the embankment and movement of the sheet pile were hardly observed, but these were confirmed in the analysis.

Table 5. Comparison of settlement (model scale)

	Settlement of levee crown	
	Analysis	Experiment
Embankment model (model 1)	15.2mm	5mm
Sheet piles are inserted to center of the liquefaction layer (model 2)	19.6mm	3mm
Sheet piles were inserted to the base layer (model 3)	12.7mm	2mm

In the experiment, since the experiment is carried out with the model container, it is considered that the lateral flow was suppressed by the restraint of the side wall of the model. Moreover, it can be inferred that thawing of embankment was insufficient and subsidence became small.

6. Conclusion

From the comparison of the effective stress analysis and the centrifugal force model experiment, the following were found.

(1) Excess pore water pressure ratio almost agreed with analysis and experiment. It can be reproduced by setting the material parameters by the result of element simulation etc.

(2) The displacement was small in the experimental value. It is necessary to consider the influence of lateral restraint by the container and embankment freezing.

(3) The penetration depth of the sheet pile influences the dissipation of pore water pressure and dominant period.

(4) Furthermore, we think that it is necessary to consider countermeasures method with improvement.

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