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CENTRIFUGE MODELING FOR SLIDING OF GENTLE SLOPE ON PARTIALLY SATURATED FINE SAND SUBJECT TO SINUSOIDAL GROUND MOTION

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ABSTRACT

This paper presents an experimental approach, based on centrifuge modeling, for investigating the dynamic response of a gentle slope on a thin layer of saturated fine sand. Laboratory equipments and centrifuge model results are presented. In addition, measured deformation are compared to corresponding numerical estimation based on Finite Difference scheme and Finn-Byrne Model

Keywords: centrifuge model, Finite difference method, dynamic response of gentle slope

INTRODUCTION

Although much attention has been paid to understanding the lateral flow of gentle slope consisting of fully liquefiable soil layers so far, particularly since the Nigatta earthquake occurred in June, 1964, relatively little effort has been devoted to improving our understanding of the dynamic response of gentle slope in areas underlain by locally liquefiable or non-liquefiable weak soils during an earthquake. Several cases in the past huge events have shown occurrence of large deformation of gentle slope that might be induced by other factors rather than liquefaction, which resulted in severe damage of gas pipes buried near the slope. O'Rourke and Palmer (1996) reported a case of damaged buried pipes in the vicinity of gentle road slope along the Balboa Blvd. that underwent permanent ground failure reaching up to 50cm during the Northridge Earthquake. **Figure 1** illustrates a cross section reflecting the situation at the Balboa Blvd. where both zones of compression and tension cracks were expressed on the surface ground that might be induced by ground failure at the locally saturated buried soil layer.

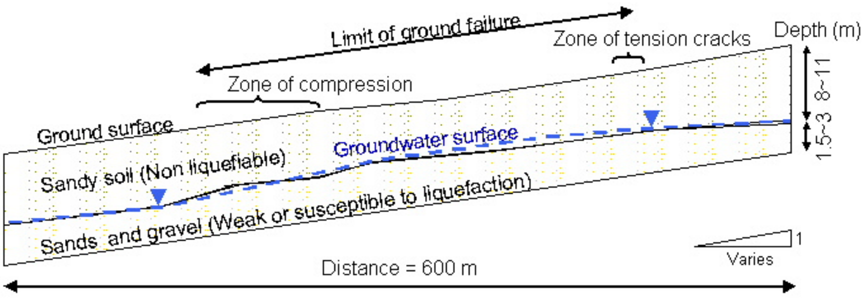

Figure 1. A typical cross section reflecting the situation near the Balboa Blvd. site

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The section profile informs us that the ground failure is suspected to have been caused by liquefaction or dynamic shear in the soil below the groundwater at depth of less than 10 meters.

By the way, Japan is well known for frequent earthquakes, which might imply quite high possibility of occurrence of permanent ground deformation causing breakage or malfunction of nearby buried pipes. Furthermore, similar to the case at Balboa Blvd, great portion of gas pipes in Japan also run along motor roads that might encounter buried weak soils and groundwater levels at certain depths which may cause local ground failures with high levels of ground shaking. Nevertheless, study on deformation of gently sloping ground underlain by partially saturated ground, where liquefaction over the entire ground is unlikely, lacking compared to the one for lateral flow induced by liquefaction over the whole ground. For investigating such deformation of a gentle slope, the authors have performed a series of centrifuge tests simulating sliding of the slope that appeared to be caused by a different mechanism than liquefaction. Again, a fully nonlinear numerical model for estimation of the dynamic response of gentle slope has been constructed on the basis of the centrifuge model. This paper addresses the outline of numerical model and its validation through the comparison between the results from centrifuge and numerical models.

BREIF DESCRIPTION OF CENTRIFUGE TEST

The examination of generation of excessive pore pressure, apart from if it grows to the threshold of liquefaction, and resulting displacements according to the intensity of seismic excitation is a major concern for this experimental approach. A rigid container carrying two different soil layers and a layer of groundwater for the centrifuge model has been manufactured and the displacement of model ground and pore pressures at several locations were monitored in accordance with applied seismic motions.

Centrifuge Facilities

Dynamic tests were carried out on the Kajima Centrifuge Version II having an in-flight radius of 2.47 m (to bucket platform) at the Kajima Technical Research Institute. The centrifuge is capable of carrying a maximum payload of 1,000kg at 100 g for static motion and 200kg with a maximum acceleration of 25 g for dynamic motion. The maximum model is 70cm long, 40cm wide and 65cm tall. A hydraulic actuator, which is able to generate a wide range of frequency from 10 to 400 Hz, is used to simulate actual earthquake motions in direction parallel to the arm in the plane of rotation. The transfer of data involving acceleration, displacement and pore pressure is enabled by 96ch of transducers. For this research, a rigid box, of which base was sloping in a gradient of 15%, with inside dimensions of 70cm long, 20cm wide and 20cm tall, was utilized to contain the model grounds.

Preparation of Experimental Model

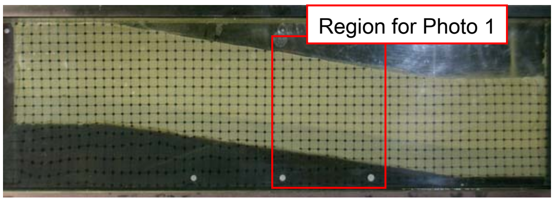
Model Configuration and Construction

Two types of model grounds have been prepared as summarized in **Table 1**. Silica sand No.8 was basically used for model ground. Kibushi clay and Silica sand No.6 were added to prepare silty sand with a mix proportion of 1:4:1 for Silica sand No.6, Silica sand No.8 and Kibushi clay respectively. Both the sand and silty sand layers were placed by pluviation and then lightly compacted to achieve the aimed relative density. Right before the placement of model ground, installed on the transparent side wall was a thin rubber membrane, whose thickness was 0.25mm, on which a grid was plotted at a spacing of 1 cm for both vertical and horizontal directions so that the relative displacement in the model ground during the tests could be tracked by movement of the grid on the membrane which had shown tight adhesion to the surface of model ground even under a centrifugal acceleration of 50 g through the preliminary verification. The groundwater table was produced along the top of the lower soil layer with the help of water tanks installed at both sides of the container, as shown in **Figure 2** which helped maintain free surface at a certain depth by controlling the quantity of water in the tanks.

Table 1. Testing conditions for centrifuge model

Soil	Thickness	Material	Dr	Saturation
Upper	8cm(4m)	Fine Sand	100%	14%
Lower	3cm(1.5m)	Silty Sand	80%	100%

() means the prototype dimension converted according to the similitude rule for centrifuge tests



Side view of model ground before loading

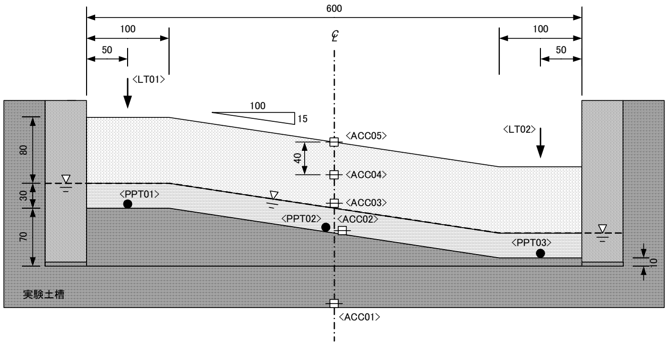


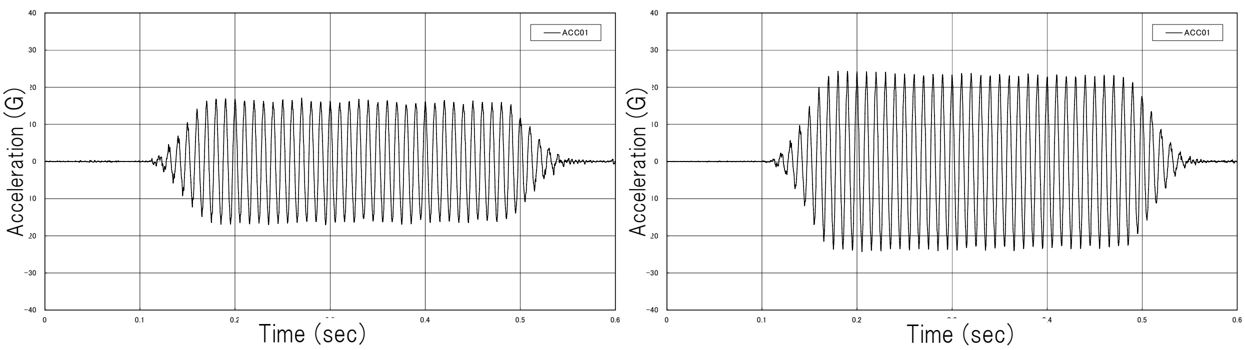
Figure 2. Longitudinal profile of sand container for centrifuge test

Input Motions and Instrumentation

Two types of sinusoidal motions were used to shake the models as shown in **Table 2** and **Figure 3**. The models were first shaken with the low-amplitude sinusoid and then shaken with the high-amplitude one to observe the response of model ground against varying intensity of input motion under a centrifugal acceleration of 50 g. The sinusoidal input motion with a peak acceleration of 24.3G and a frequency of 2Hz has been employed to the bottom of the container under a centrifugal acceleration of 50G. Accelerators and pressure meters were installed in the model ground to monitor the amplified acceleration and the excessive pore water pressure in the saturated fine sand during shaking as shown in **Figure 2**. Displacement at the surface of model ground was recorded near the top and toe of the slope using laser displacement transducers to monitor the movement of surface ground accompanied by lateral deformation of the slope. **Table 3** presents the summary of instrumentation during the tests.

Table 2. Summary of input motions

Item	Test 1	Test 2
Maximum amplitude	16.9G(9th cycle)	24.2G(10th cycle)
Average amplitude for a range from 15th cycle to 35th cycle	16.2G	23.3G
Relative intensity with reference to the input motion for Test 2	70%	100%
Centrifugal acceleration	50G	50G



(a) For Test 1 **(b) For Test 2**

Figure 3. Sinusoidal seismic waves for centrifuge models (obtained from ACC01)

Table 3. Items for monitoring during centrifuge tests

Monitoring Items	Monitoring cell	Number of Cells	Acronym	Indicator
Response of ground acceleration	Accelerometer	5	ACC	⊟
Pore water pressure	Pore pressure transducer	3	PPT	○
Vertical displacement at top surface of mode ground	Laser displacement transducer	2	LT	↓
Displacement in model ground (Vertical and horizontal directions)	Numerical processing of images captured by video camera	-	-	-

Experimental Results and Discussions

Deformation of model ground

Photo 1 shows the side view of testing box after the sinusoidal loadings had been applied. Relative movement of markers indicated that the deformation was concentrated at the lower silty sand layer and the magnitude of deformation appeared to become larger as the amplitude of input wave was increased. **Figure 4** illustrates the distribution of maximum shear strains that was calculated from the observed relative displacement at each marker. **Table 4** summarizes the average maximum shear strains generated by the sinusoidal motions during the tests regarding the area corresponding to each soil layer, where the upper sand layer was subdivided by two regions over the depth. As shown in **Table 2**, the results from both the testing cases indicated that shear deformation mostly took place at the lower silty sand in which generation of excessive pore water pressure was expected, and no significant shear deformation occurred near the top surface of the model ground. This observation thus implied that the horizontal displacement of the entire model ground was mainly associated to the shear deformation of the saturated silty sand layer. **Figure 5** shows the distribution of horizontal displacements in the model ground that were monitored while seismic loading was activated. As shown in **Figure 5**, the model for Test 2 exhibiting the maximum horizontal displacement of 5.0cm, which was executed with the stronger sinusoidal motion, produced horizontal displacements greater than those from the model for Test 1 that presented the maximum horizontal displacement of 2.8mm. Further, **Figure 5** demonstrates that the observed horizontal displacements grew remarkably at the depth from 0 (the bottom of model) to 30 mm corresponding to the saturated silty sand layer, whereas their increment appeared as being moderate apparently at the depth above 40 mm that corresponded to the dry sand layer, which provided insight into how the gently sloping ground underlain by a saturated weak soil behaved while being subjected to different levels of seismic motion.

Table 4. Average maximum shear strains developed in the model ground

Case	Upper sand layer – dry		Lower silty sand layer - Saturated
	Near the surface	Middle of the ground	
Test 1	0.5%	1.0%	7.4%(18.0% for peak value)
Test 2	-2.7%	0.8%	15.3%(24.6% for peak value)

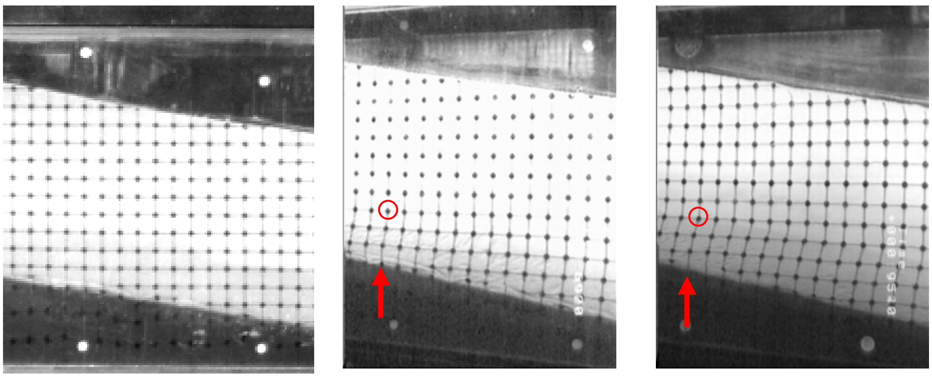
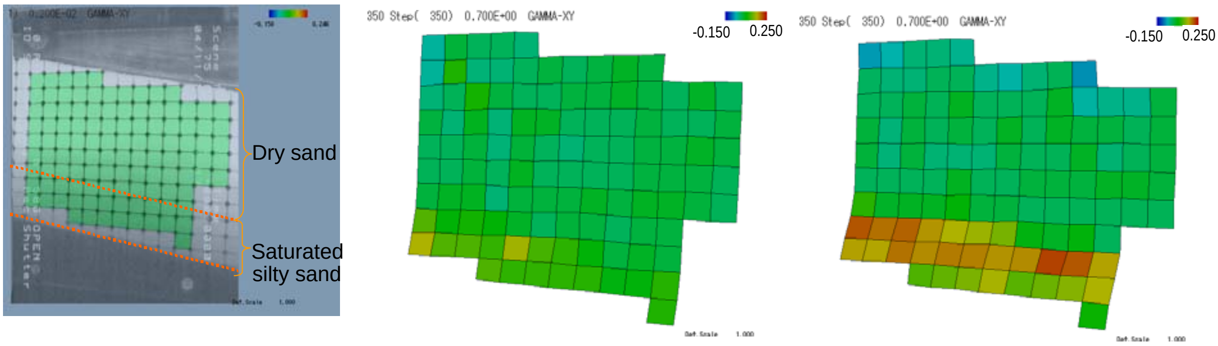
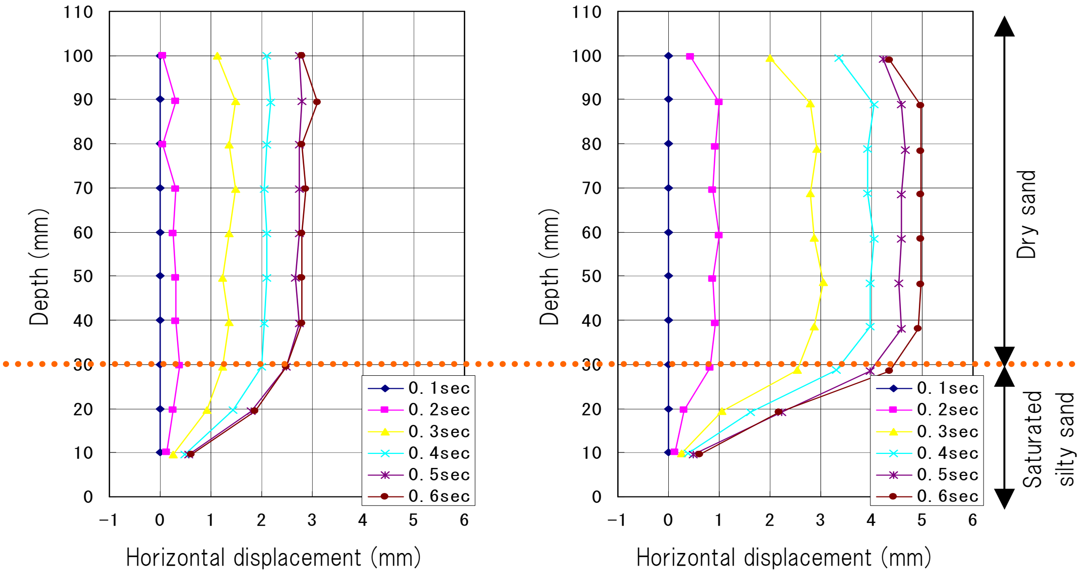


Photo 1 Deformed ground after seismic loading (Leftmost: Before test, Middle: Test 1, Rightmost: Test 2)



(a) Before the tests (b) Test 1 after seismic excitation (b) Test 2 after seismic excitation

Figure 4 Maximum shear strains corresponding to deformation of model ground



(a) Test 1 (b) Test 2

Figure 5. Vertical distribution of horizontal displacement at the middle of slope from centrifuge model

Thus, it could be acquired from the observation on deformation of the models that the dynamic response of a gentle slope involving a groundwater table at a certain depth would be governed mainly by shear deformation of the ground below the groundwater table even with varying intensity of seismic motions.

On the other hand, although the horizontal displacements near the surface of model ground was seen to draw back slightly in **Figure 5**, which might be misread due to lack of adhesion between the rubber membrane and model ground, they appeared to be almost uniform at the depth from 40 to 90mm within the upper sand layer for both the two testing cases. **Table 5** provides the average horizontal displacement for that range of depth according to the elapsed time of sinusoidal loading. Whereas the ratio for the maximum amplitude of input motions - (Max. amplitude of Test 1) / (Max. amplitude of Test 2) - was recorded as 70% as described in **Table 2**, the ratio of average horizontal displacement for the two testing cases was observed as much as 58%, which indicated that ground deformation might grow much severely in relation to incremental seismic intensity.

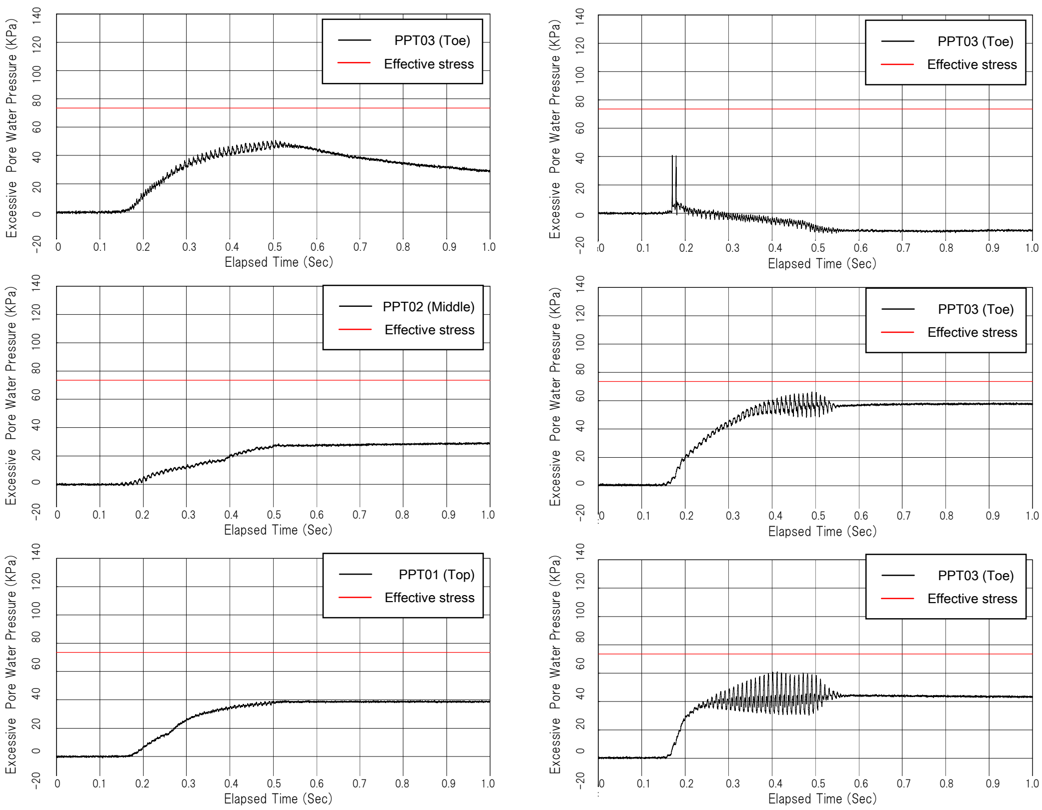
Table 5. Average horizontal displacement and ratio of the results from Test 1 and Test 2

	0.1sec	0.2sec	0.3sec	0.4sec	0.5sec	0.6sec
Test 1	0.00 mm	0.26 mm	1.39 mm	2.10 mm	2.74 mm	2.86 mm
Test 2	0.00 mm	0.93 mm	2.88 mm	3.98 mm	4.60 mm	4.96 mm
(Test 1)/(Test 2)	-	28%	48%	53%	59%	58%

Generation of excessive pore pressure

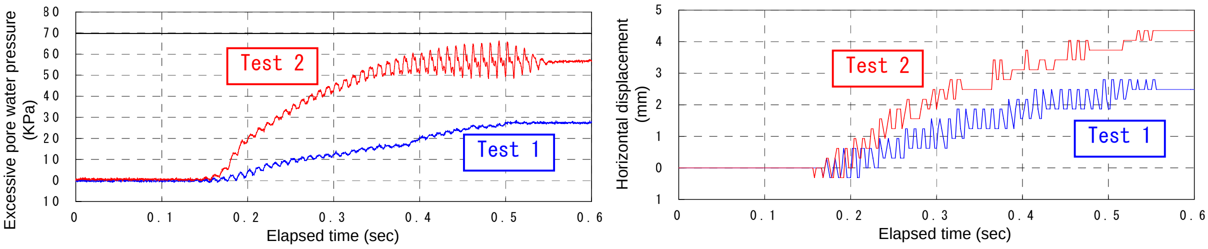
Figure 6 shows the time histories of excessive pore water at the three monitoring locations as guided in **Figure 2**. All the histories except for the PPT03 for Test 2, which was located at the top of the slope, exhibited gradual increase of excessive pore water pressure from the time when the amplitude of input motion reached its peak value (approximately 0.16sec in **Figure 3**). The PPT03 for Test 2 failed to record the data properly in the course of test for uncertain mechanical trouble. Although the observation of pore water pressure meters during shaking identified a gradual rise of pore water pressures at the bottom of the saturated silty sand layer, they appeared to remain below the threshold of liquefaction which was indicated by red line in **Figure 6** at any case. Again, similar to the displacement of model ground, the model for Test 2 had generated excessive pore water pressure greater than that for Test 1.

Figure 7 shows the time histories of excessive pore water pressure and horizontal displacement at the middle of the slope, of which monitoring locations were indicated by ↑ for pore water pressure and ○ for horizontal displacement in **Figure 4**, for both two testing cases. As for Test 1, the excessive pore water pressure appeared to rise gradually as the sinusoidal loading continued and reached its peak value at 0.5 sec in the elapsed time. However, the peak value for observed pore water pressure was much less than the initial vertical effective stress at the middle of the slope exhibiting 70KPa. Similarly, the horizontal displacement for Test 1 was observed to grow gradually as the excessive pore water pressure got increased. Besides, the excessive pore water pressure for Test 2 was observed to increase more quickly than that for Test 1 at early time of the dynamic loading resulting in its peak value of 66KPa at the elapsed time of 0.4 sec and then maintain almost constant value. Meanwhile, the horizontal displacement for Test 2 appeared to grow in a fashion that was similar to that for the Test 1, even though its peak value was greater than that of Test 1. It could be thus acquired without doubt that the deformation of slope underlain by a saturated soil layer was mainly associated to the degree of excessive pore water pressure developing during a dynamic loading that led to lessening of vertical effective stress.



(a) Test 1 **(b) Test 2**

Figure 6. Time histories of pore water pressure



(a) Excessive Pore water pressure **(b) Horizontal displacement**

Figure 7. Time histories of pore water pressure and horizontal displacement at the middle of slope

NUMERICAL MODELING OF CENTRIFUGE TEST

The centrifuge model for Test 2 was analyzed with a two dimensional finite difference analysis program [Itasca Consulting Group, Inc. FLAC(Fast Lagrangian Analysis of Continua), Version 4] in conjunction with the Finn-Byrne suggestion for generation of excessive pore water pressure during a shaking motion. Using the FLAC, fully nonlinear dynamic analyses that were fully coupled including mechanical aspects and groundwater flow, have been performed. To evaluate the influence of excessive pore pressure generated in the fully saturated silty sand layer on the seismic behavior of entire slope, two kinds of numerical models with/without generation of excessive pore pressure during seismic excitation have been compared with respect to the maximum lateral displacements of entire slope.

Numerical models and input parameters

To evaluate the effect of excessive pore water pressure on the response of the slope, two dynamic simulations have been carried out according to the generation of pore water pressure or not by the switch to the activation of FINN-BYRNE model that enables generation of excessive pore water pressure during a ground motion using Eqn. (1).

$$\Delta\varepsilon_{vd}/\gamma = C_1 \exp(-C_2(\varepsilon_{vd}/\gamma)) \qquad (1)$$

where, $\Delta\varepsilon_{vd}$= the increment of volume decrease,

ε_{vd}= the accumulated irrecoverable volume strain,

γ = the cyclic shear-strain amplitude,

C_1, C_2 = the constants that can be derived from relative densities of sandy soil, Dr

Namely, $C_1 = 7600(D_r)^{-2.5}$, $C_2 = \dfrac{0.4}{C_1}$ (Byrne, 1991)

The numerical model was constructed in a dimension same as the centrifuge model as shown in **Figure 8**. Elasto-perfectly plastic material based on the Mohr-coulomb failure criterion was assigned to two soil layers and elastic material was to the foundation. To avoid the distortion of plane wave propagating upward at the boundary, the model for FLAC employed the free field boundary for both its sides that encouraged involvement of the engineering foundation to make the bottom of the model flat (Itasca, 2005). The acceleration histories monitored at ACC01, of which peak acceleration was 456.6 Gal at the dominant frequency of 2Hz (100Hz in the scale of centrifuge model), from the Test 2 was set to the input waves for numerical analysis after being processed with a trapezoidal band pass filter for baseline correction. The duration was approximately 25 sec (0.5 sec in the scale of centrifuge model). The Rayleigh damping of 10% had been joined to the fully nonlinear analysis with the dominant frequency of 2 Hz. The input parameters for both the cases were specified as **Table 6**, corresponding to Mohr-Coulomb materials.

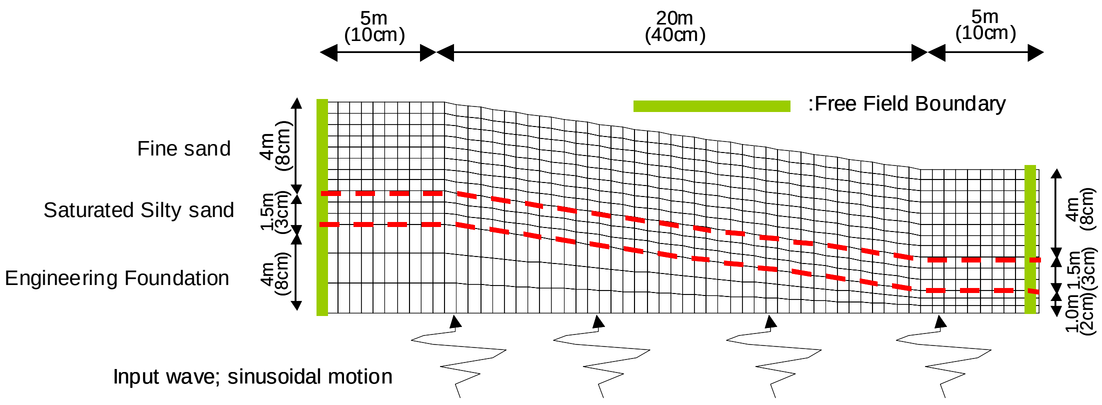


Figure 8. Model for numerical analysis

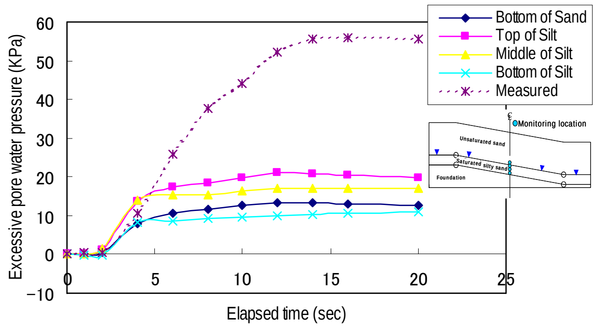
Table 6. Input Parameters for fully nonlinear analysis using FLAC

Soil	Dry density (t/m^3)	Shear Modulus (KPa)	Bulk Modulus (KPa)	S-wave velocity (m/sec)	Damping ratio	Cohesion (KPa)	Friction angle (°)	Poisson's ratio	FINN-BYRNE coefficient	
									C1	C2
Unsaturated Sand (upper)	1.874	7.58E+04	1.85E+05	201	10%	5.0	40.0	0.32	0.086	4.625
Saturated Sand (Lower)	1.512	4.22E+04	1.27E+05	167		11.3	33.9	0.35	0.764	0.523
Engineering Foundation	2.700	1.00E+06	1.67E+06	609		-	-	0.25	-	-

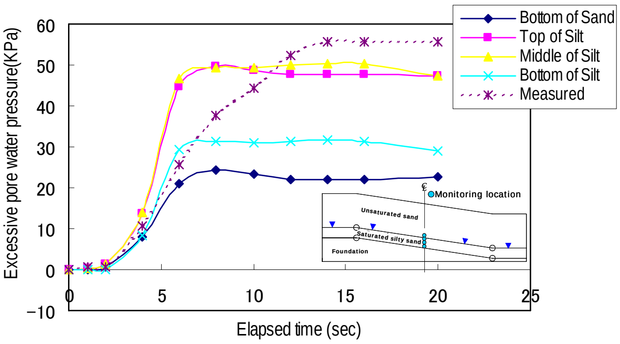
Occurrence of excessive pore water pressure

Figure 9 shows the calculated excessive pore water pressure histories at the bottom of saturated silty sand in the middle of the slope for the model without/with the FINN-BYRNE model respectively during the sinusoidal excitement. It was assumed in the model without Finn-Byrne model that no fluid flow occurred and that no pore pressure generation occurred due to particle rearrangement. Even though slight lifts of pore water pressures are seen even in **Figure 9 (a)** (without FINN-BYRNE model) because of the dynamic volume changes induced by the sinusoidal excitement, they showed little change after 4 seconds in the elapsed time.

In the mean time, despite the observation that growing rates of excessive pore water pressures in the numerical calculation appeared as faster than those of the centrifuge model in the early time of sinusoidal excitement, the numerical model with FINN-BYRNE suggestion exhibited the remarkable rises of pore water pressures up to approximately 50 KPa, equivalent to a pore water pressure ratio (u/σ) of 0.6, as being shaken, which indicates a good match with the results from the centrifuge model. This implies that the significant increases of pore water pressures in the saturated silty sand lead to the reduction of effective stresses in the soil layer, which may result in yielding of the soil elements and thereby plastic flow at the soil layer.



(a) Without FINN-BYNRE model



(b) With FINN-BYNRE model

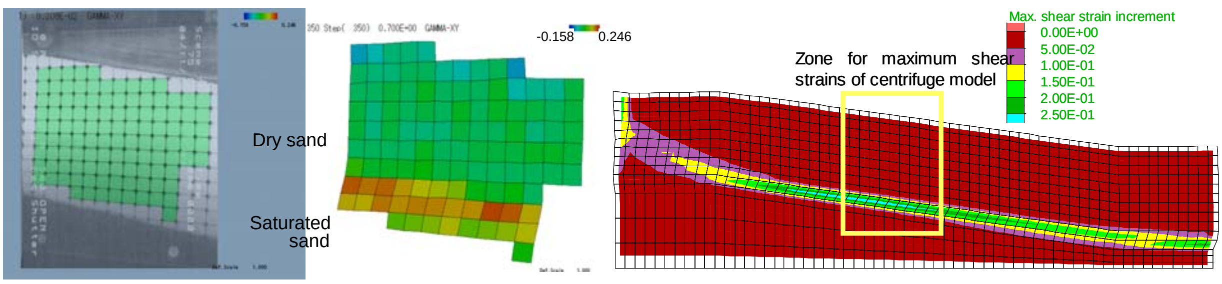
Figure 9. Histories of pore pressures during seismic loading

Ground deformation induced by shaking

Figure 10 presents the distribution of maximum shear strains at the elapsed time of 25 seconds, which is equivalent to 0.5 sec in the centrifuge model, for the centrifuge and numerical (with FINN-BYRNE model) model respectively. Both the models clearly demonstrated the concentrated shear deformation in the saturated silty sand, which brought about the sliding of entire area of unsaturated sand. The maximum value of the maximum shear strains appeared as approximately 25% for either of them, which indicated that the current numerical model was able to reasonably simulate the behavior of gentle slope being shaken at the centrifuge model.

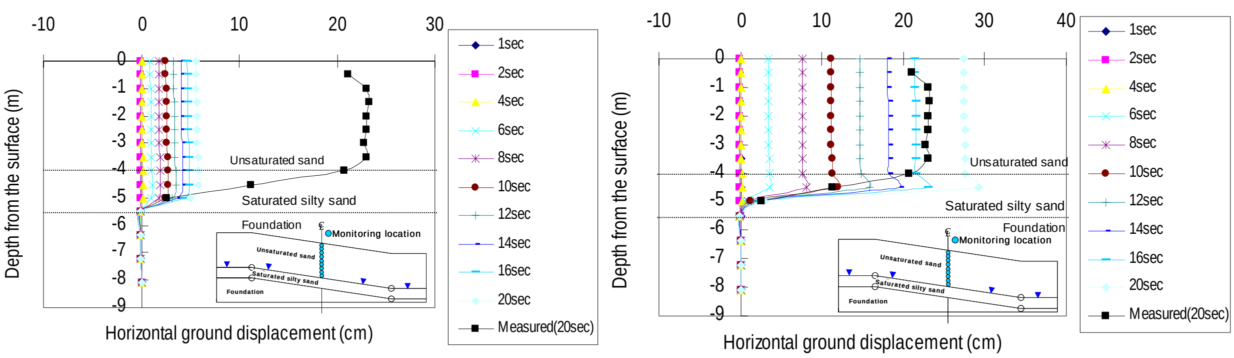
By the way, **Figure 11** provides the comparison between the centrifuge and numerical model with respect to the calculated displacement according to the adoption of FINN-BYRNE model. In spite of slight growth of the pore water pressures as shown in **Figure 9 (a)**, the model without the FINN-BYRNE model began to generate the horizontal displacement in relatively small amounts since the sinusoidal excitement had been loaded and resulted in no more than approximately 6cm (1.2mm for centrifuge model) which was much less than the measured one in the centrifuge test. Meanwhile, as expected, the model with FINN-BYRNE model exhibited gradual increase of horizontal displacement as the sinusoidal loading continued, which provided a reasonable match with the results from the centrifuge models.

Given that the only difference between the two models was whether the model could generate excessive pore water pressure induced by particle rearrangement during seismic excitation, an importance issue for predicting dynamic behavior of gentle slope underlain by a saturated sandy soil should be adequate generation of the excessive pore water pressure that causes the effective stresses to go to zero and thereby the shear strength to reduce to a strain-mobilized shear strength.



(a) Centrifuge model for Test 2 **(b) Numerical model with Finn-Byrne model**

Figure 10. Maximum shear strains at the numerical model and centrifuge model



(a) Without FINN-BYNRE model **(b) With FINN-BYNRE model**

Figure 11. Calculated horizontal displacements at numerical model

CONCLUSIONS

The dynamic response of gentle slope underlain by a layer of saturated silty sand subject to strong ground motion has been quantitatively investigated through both the centrifuge models and the numerical models based on finite difference scheme in this study. The findings gained from this study are summarized as follows;

- The centrifuge models subjected to the two sinusoidal input motions varying in intensity have quantitatively identified the incremental lateral deformation of gentle slope in accordance with the increase of intensity of seismic excitation.

- The observation on development of excessive pore water pressure within the silty sand being located below the groundwater table has clearly shown that significant rises of excessive pore water pressure can cause the effective stresses in the soil layer to reduce to a extremely low level that may result in yielding of the soil elements and thereby plastic flow at the soil layer.

- The comparison between the centrifuge and numerical model has clearly shown that the fully nonlinear elasto-plastic model based on the Mohr-Coulomb failure criterion incorporated with FINN-BYRNE's suggestion has shown a reasonable match with the results from centrifuge model with respect to both the pore water pressure and the deformation of entire slope.

- Accordingly, it can be acquired from this study that even a gentle slope may cause a great degree of lateral ground deformation when subjected to high levels of ground shaking if the slope encounters weak soil layers involving groundwater level even at a certain depth from the surface of the slope.

In addition a total stress analysis involving shear modulus reduction and hysteretic damping under cyclic loadings for the saturated silty sand will be successively conducted for further study.

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