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MODELING OF DRAINAGE BEHAVIOR FOR DYNAMIC EFFECTIVE STRESS ANALYSIS BY UNDRAINED CONDITION

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SUMMARY

A method to consider excess pore water pressure dissipation in high permeable ground, such as gravel drains installed as remedial measures against liquefaction, is proposed in the dynamic effective stress analysis with undrained condition. Effect of the drainage is considered by increasing apparent liquefaction resistance so that the excess pore water pressure in the high permeable ground becomes same with the model under undrained condition. The increase of the liquefaction resistance is evaluated by cumulative damage concept. The cumulative damage at improved ground is obtained by the consolidation analysis subjected to the same stress cycles with unimproved ground. Large-scale model tests were simulated. Responses such as acceleration, displacement and excess pore water pressure show good agreement with the test results.

INTRODUCTION

In resent years dynamic effective stress analysis has been becoming a powerful technique to evaluate the liquefaction-induced damage to structures. It is also useful to evaluate efficiency of remediation measures against liquefaction. Many computer codes have been developed. Some codes consider seepage of excess pore water pressure by Biot's or Terzaghi's equation. However some do not because duration of the earthquake is too short for excess pore water pressure to dissipate. In the latter case it seems efficient to

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analyze high permeable ground such as gravel drains installed as remedial measures against soil liquefaction. In this paper the method to consider the high permeable ground by effective stress analysis under undrained condition is proposed.

MODELING OF DRAINAGE EFFECT

Liquefaction remediation measures based on excess pore water pressure dissipation have an effect to restrain the excess pore water pressure generation with high permeable material installed in the ground. Gravel piles installed in the ground keep excess pore water pressure less than initial confining stress. The cumulative damage concept that has been used for the design of the gravel drain method is extensively applied to evaluate this effect. The idea is that effect of drainage to keep excess pore water pressure low is replaced by high increasing liquefaction resistance of model. The procedure to model the drainage effect using the cumulative damage concept is shown in Figure 1.

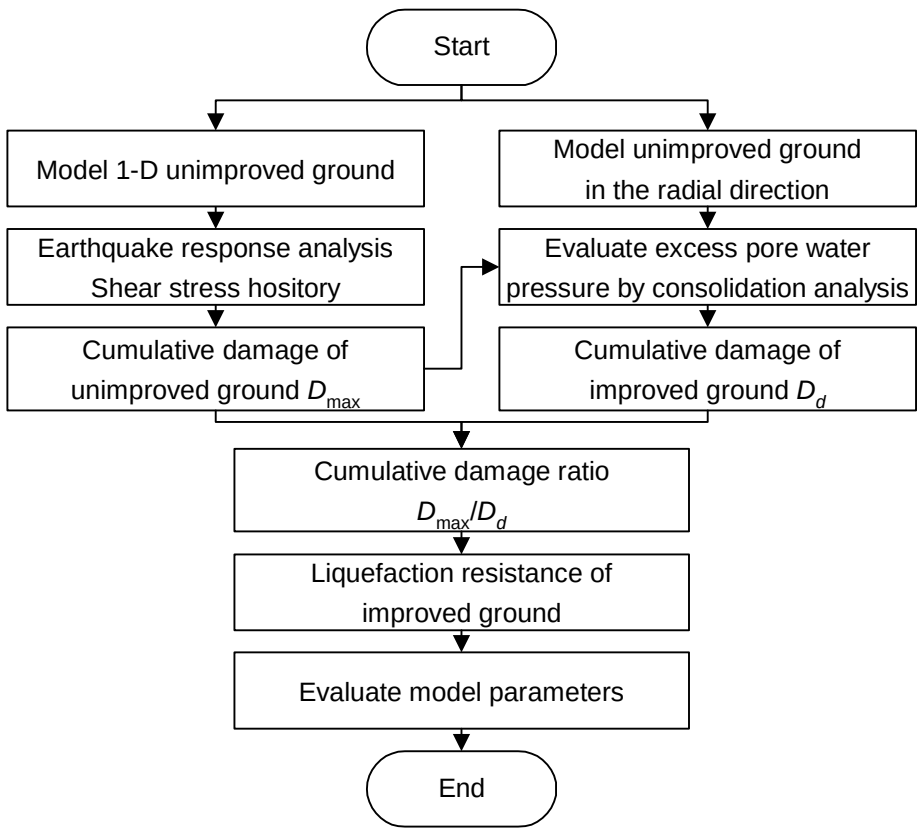

Figure 1 Procedure to model the drainage effect

Increase of the liquefaction resistance in the model is evaluated from the decrease of the cumulative damage, which means that the liquefaction resistance curve of the improved ground moves to the right depending on the cumulative damage ratio, as shown in Figure 2. The cumulative damage ratio is calculated by

$$\delta = \frac{D_{\max}}{D_d} \tag{1}$$

in which $D_{\max}$ is a cumulative damage in the unimproved ground, and D_d is an apparent cumulative damage of the improved ground. Apparent cumulative damage D_d should be reduced below $D_{\max}$ if the drainage effect is considered. The cumulative damage is calculated by

$$D = \frac{1}{2}\sum_k \left[\frac{1}{(N_l)_k} \right] \tag{2}$$

in which $(N_l)_k$ is the number of the k-th shear stress pulse within the shear stress history required to cause liquefaction as schematically shown in Figure 3. The cumulative damage D is unit when the soil liquefies. $D_{\max}$ is calculated by Eq. (2) using the shear stress history in the unimproved ground that is obtained by an earthquake response analysis.

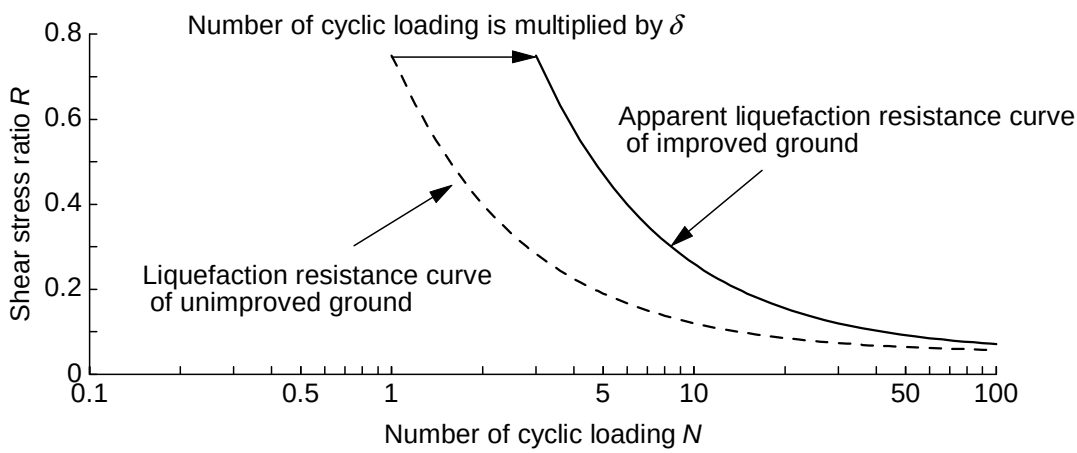

Figure 2 Liquefaction resistance of the improved ground

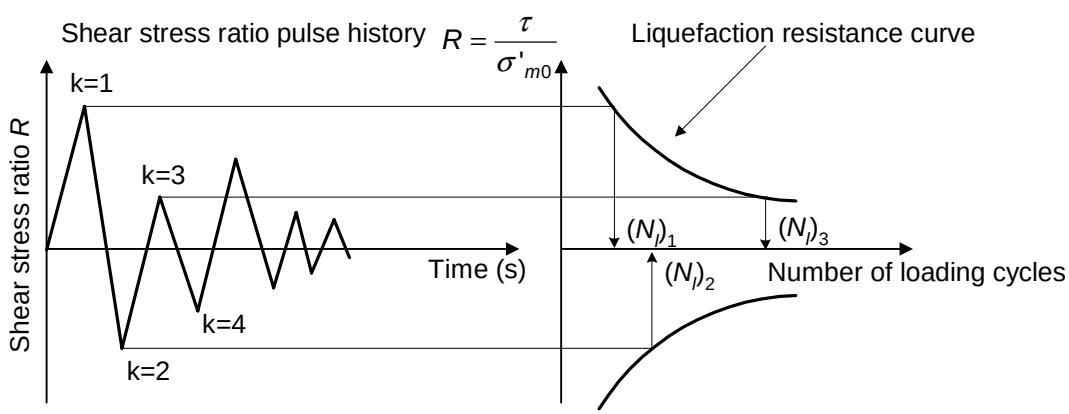

Figure 3 Calculation of the cumulative damage

On the other hand the drainage effect is evaluated by a consolidation analysis. Generation of the excess pore water pressure is assumed to be same with that in the unimproved ground. Under the conditions of purely radial drainage to the gravel piles, the consolidation equation is lead to the following form [1]

$$\frac{k_h}{\gamma_w m_v}\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}\right)=\frac{\partial u}{\partial t}-\frac{\partial u_g}{\partial N}\frac{\partial N}{\partial t} \tag{3}$$

in which u is the excess pore water pressure; k_h is a coefficients of permeability; γ_w is the unit weight of water; m_v is a coefficient of volume compressibility; u_g is excess pore pressure generated during the dN loading cycles. For the practical purpose, the irregular cyclic loading is converted to an equivalent number N_{eq} of uniform stress cycles at a stress ratio τ/σ'_{v0} occurred in the duration of t_d. Then,

$$\frac{\partial N}{\partial t}=\frac{N_{eq}}{t_d} \tag{4}$$

Relationships between u_g and the number of cycles N is evaluated in the following [1]

$$\frac{u_g}{\sigma'_{v0}}=\frac{2}{\pi}\arcsin\left(\frac{N}{N_l}\right)^{1/2\alpha} \tag{5}$$

in which u_g/σ'_{v0} is an excess pore water pressure ratio; N_l is the number of alternating shear stress cycles required to cause liquefaction under certain stress amplitude. The shape of excess pore water pressure generation curve derived from Eq. (5) is shown in Figure 4. A finite element program LARF (Liquefaction Analysis for Radial Flow) [1] is used to solve Eq. (3). Even if the number of alternating shear stress cycles, N, extends to N_l, u_g/σ'_{v0} is kept 1.0. In this sense, N/N_l can be considered to be same with D_d in Eq. (5). By replacing N/N_l with D_d in Eq. (5), the apparent cumulative damage in the unimproved ground yields

$$D_d=\sin\left(\frac{\pi}{2}\cdot\frac{u_g}{\sigma'_{v0}}\right)^{2\alpha} \tag{6}$$

Finally the element simulations are performed to find the model parameters targeting the liquefaction resistance of the improved ground obtained by the procedure in Fig.1. In addition the effect of rigidity of the gravel pile is examined later, which can become dominant when the excess pore water pressure is high in the unimproved ground.

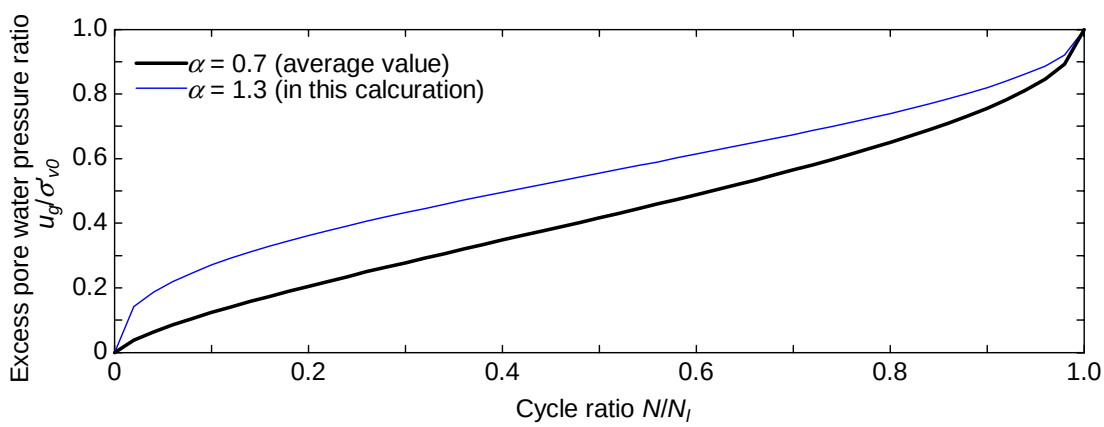

Figure 4 Relationships between u_g and the number of cycles N

DYNAMIC EFFECTIVE STRESS ANALYSIS PROGRAM

A computer program code FLIP (Finite element analysis of LIquefaction Program) [2] is used in this study. The constitutive model is based on a multiple inelastic shear springs defined in the deviator strain space as shown in Figure 5 originally proposed by Towhata [3]. In addition to the conventional assumption of hyperbolic relationship assigned for each shear spring for monotonic initial loading, the model uses the extended Masing rule to reproduce more realistic hysteresis loop for cyclic loading [4, 5 and 6]. The excess pore water pressure generation due to dilatancy is modeled using the concept of liquefaction front, which is defined in the equivalent normalized stress space, shown in Figure 6 [2]. This is based on the results of the laboratory tests by Towhata [7] that there is a unique relationship between the accumulated shear work per unit volume and the excess pore water pressure at each state of shear stress. In Figure 6, the coordinate S is equivalent to a ratio of current over initial confining stresses and the coordinate r is equivalent to deviatoric stress ratio. The normalized excess pore water pressure is given by 1-S. The liquefaction front specifies the current value of S as a function of the liquefaction front parameter S_0, which is directly specified as a function of normalized plastic shear work while the parameter S, equivalent to the confining stress ratio, is specified by the liquefaction front curve. If the deviatoric stress ratio r becomes larger than m_3, the parameter S will be affected by the effect of positive dilatancy. The main ten parameters used in the constitutive model are shown in Table 1.

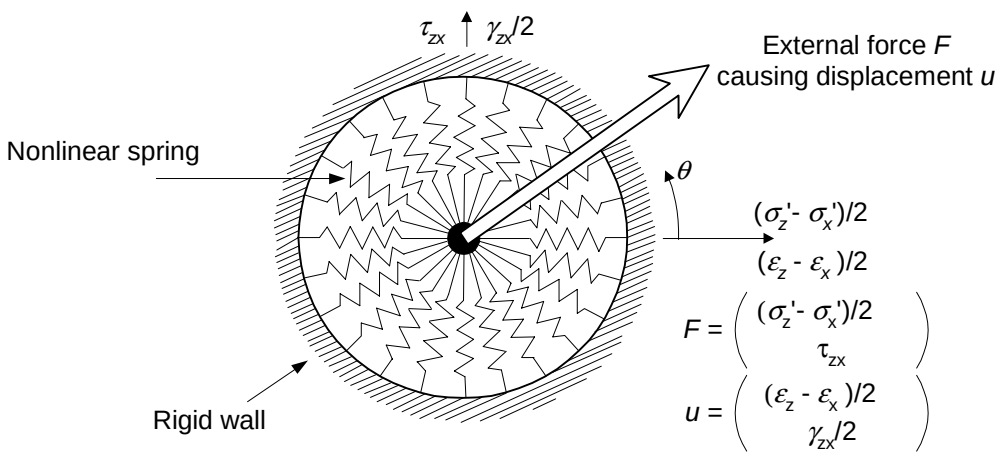

Figure 5 Multiple shear spring model (after Towhata and Ishihara [3])

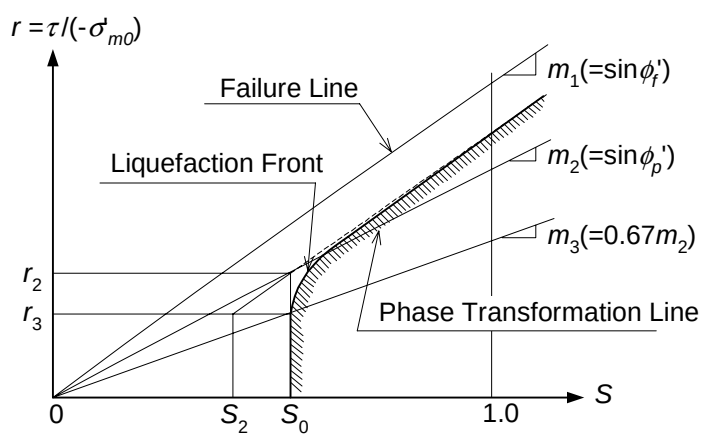

Figure 6 Schematic figure of liquefaction front (after Iai et al. [2])

Table 1 List of model parameters

Parameters	
K_{ma}	Rebound modulus
G_{ma}	Shear modulus
ϕ_f	Shear resistance angle
h_{max}	Hysteresis damping factor at large shear strain level
ϕ_p	Phase transformation angle
w_1	Parameter to control overall e.p.w.p generation
p_1	Parameter to control initial phase of e.p.w.p generation
p_2	Parameter to control final phase of e.p.w.p generation
S_1	Minimum effective confining stress ratio
c_1	Threshold limit to cause dilatancy

SIMULATION OF LARGE-SCALE MODEL TESTS

Large-scale model tests

The large-scale model tests [8] are simulated. A container made of a stack of 64 aluminum rings is set on the shaking table. Rings are 1.9 meters in diameter, and are stacked to the height of about two meters. The cross sectional geometric configuration is shown in Figure 7. The composite ground is made of two parts; one part is the sand deposit, with relative density of about 30% and the other the gravel drain. The sand was taken at Gaikou district in Akita Port, Japan. Fines were completely washed out for securing the uniformity in permeability. Grain size distributions are shown in Figure 8. Two kinds of crushed stones were used for the gravel drain. No.7 crushed stones were used for the gravel piles. No.5 crushed stones were for the gravel mat of 20 cm thickness above the sand deposit and the gravel drain.

Test series for the numerical simulation are shown in Table.2. The tests were conducted under sinusoidal input motion. Horizontal acceleration, and horizontal displacement, and excess pore water pressure at the point of A13, H13, and P23 shown in Figure 7 are compared with simulation.

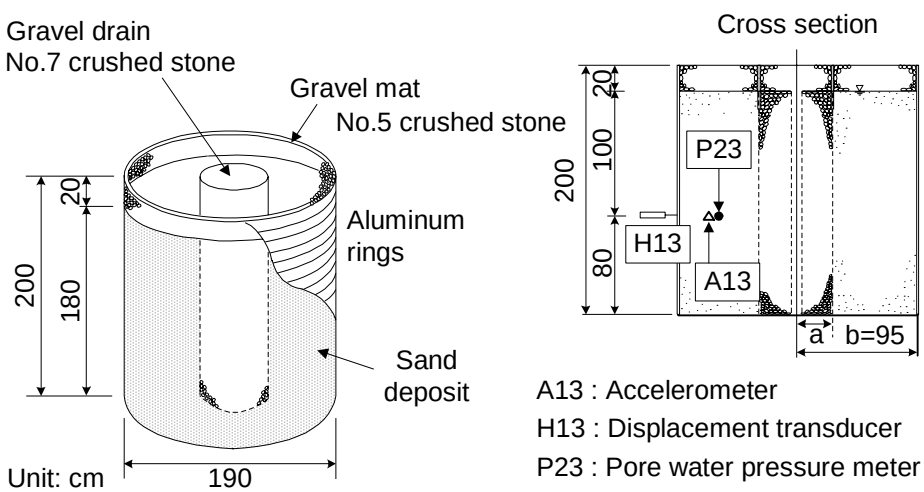

Figure 7 Test apparatus

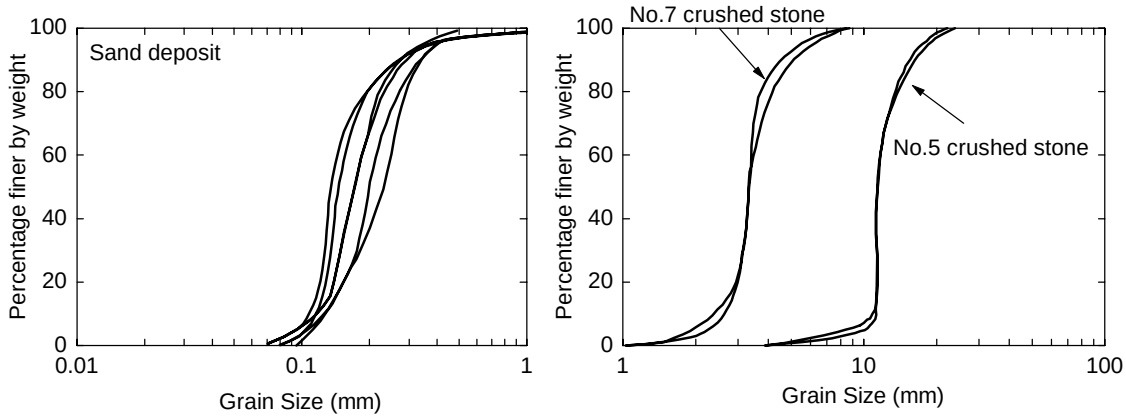

Figure 8 Grain size distribution of materials

Table 2 Test series for the numerical simulation

Test cases	Pile diameter ratio	Conditions of shaking			
	a / b	Wave type	Frequency	Duration	Input acceleration
R-502	0.30	Sinusoidal	2Hz	10 sec	49.7 m/s^2
R-702	0.45	Sinusoidal	2Hz	10 sec	51.0 m/s^2
R-703	0.45	Sinusoidal	2Hz	10 sec	81.6 m/s^2

Finite element model and model parameters

One dimensional model composed of the sand deposit and the gravel mat is modeled as shown in Figure 9. The fundamental model parameters are shown in Tables 3 and 4. They are decided based on various tests made in reference [8]. The shear modulus and the rebound modulus vary depending on confining stress. Liquefaction resistance by the analysis is compared with the triaxial test result in Figure 10. Unfortunately, the sand used for the triaxial test was not exactly same as the sand used for the shaking tests; the fines less than 5% are included in the sand for the triaxial test. Triaxial compression test results under consolidated drained condition by Ootaka [9] were used to decide the model parameters of gravel.

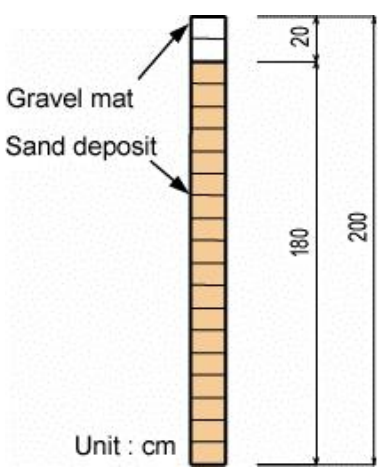

Figure 9 Finite element mesh

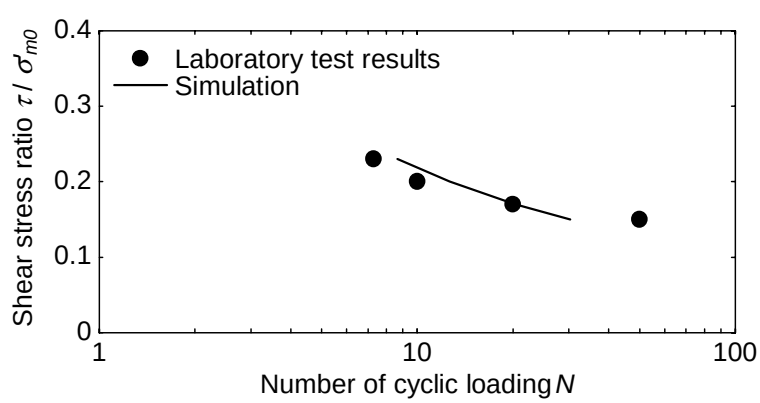

Figure 10 Element simulation test result

Table 3 Model parameters for hyperbolic nonlinear model

Materials	ρ_t	n	G_{ma} (kPa)	K_{ma} (kPa)	σ'_{ma} (kPa)	ϕ_f (deg.)	c (kPa)	h_{max}
Sand	1.76	0.55	11600	30200	6.32	38.2	0.0	0.24
No.5 gravel	1.42	0.47	1656	4318	0.93	45.4	6.9	0.24
No.7 gravel	1.89	0.46	4572	11923	7.09	45.4	6.9	0.24

* The value of G_{ma} and K_{ma} are defined at confining pressure σ'_{ma}.

Table 4 Model parameters for excess pore water pressure model

Materials	ϕ_p (deg.)	w_1	p_1	p_2	S_1	c_1
Sand	28.0	6.5	0.30	0.90	0.005	1.30

The parameters used in the consolidation analysis by LARF are shown in Table 5. The cumulative damage ratios for the test series are shown in Table 6. The simulations targeting the apparent liquefaction resistance are shown in Figures11, 12 and 13. Only the parameter w_1, which controls the overall excess pore water pressure, was changed to move the simulated liquefaction resistance curve toward the targets.

Table 5 Parameters used for consolidation analysis

Materials	k (m/s)	m_v (m^2/kN)	α
Sand	0.000092	0.00018	1.3

Table 6 Cumulative damage ratios

Test cases	Cumulative damage		Cumulative damage ratio
	D_{max}	D_d	
R-502	0.64	0.20	3.1
R-702	0.77	0.14	5.7
R-703	2.50	1.00	2.5

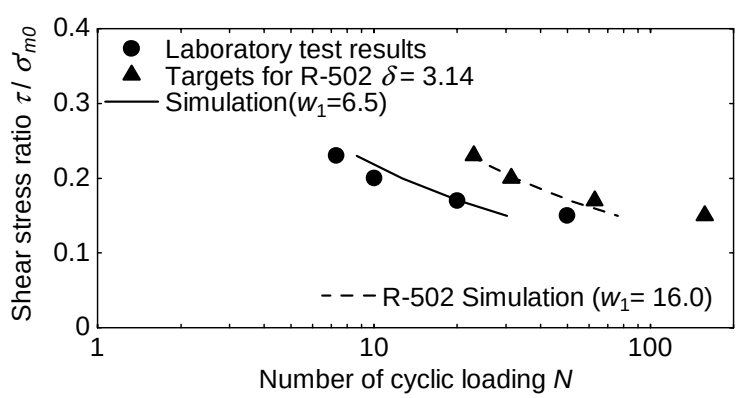

Figure 11 Element simulation test result (R-502)

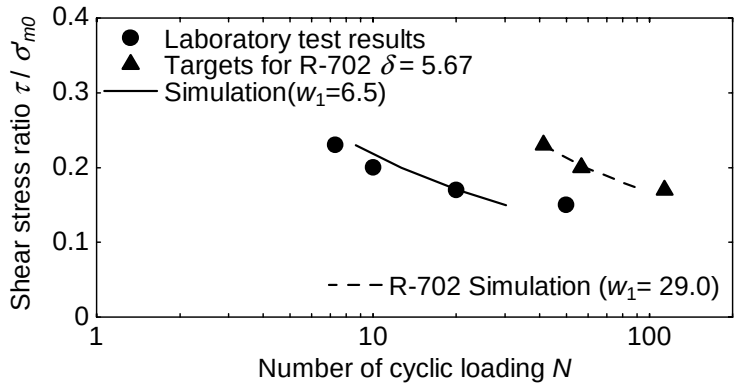

Figure 12 Element simulation test result (R-702)

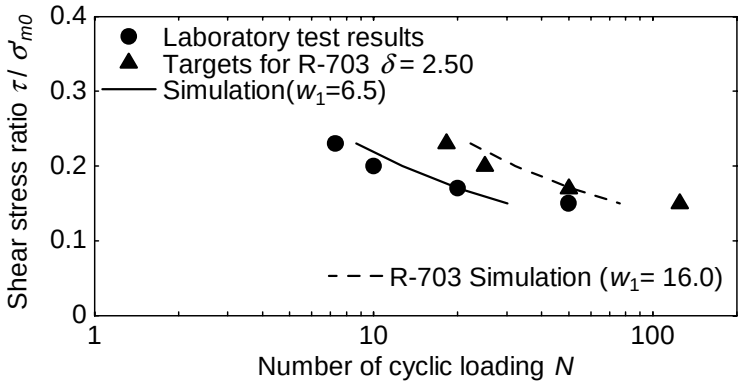

Figure 13 Element simulation test result (R-703)

RESULTS AND DISCUSSION

Responses of horizontal acceleration, horizontal displacement, and excess pore water pressure ratio are compared with the test results in Figures14, 15 and16. The excess pore water pressure generations are restrained by the drainage effect in the test cases of R-502 and R-702. The responses obtained by the simulation show good agreement with the tests. The horizontal displacements by the tests seem to be unstable. It is possibly caused by behavior of container made of aluminum rings. The simulation gives slightly rapid buildup of the excess pore water pressure at the beginning of the shaking. The reason seems to come from the difference between the sand used for the triaxial test and the sand used for shaking tests.

The sand deposit liquefies at about 7 seconds in the test R703. The generation of the excess pore water pressure is rapid in the analysis as well. In order to examine the effect of the rigidity of the gravel pile, two dimensional analysis was performed. The results are shown in Figure 17. As seen in the excess pore water pressure response the generation of the excess pore water pressure is restrained by considering the gravel stiffness, results in better agreement with the test than previous analysis.

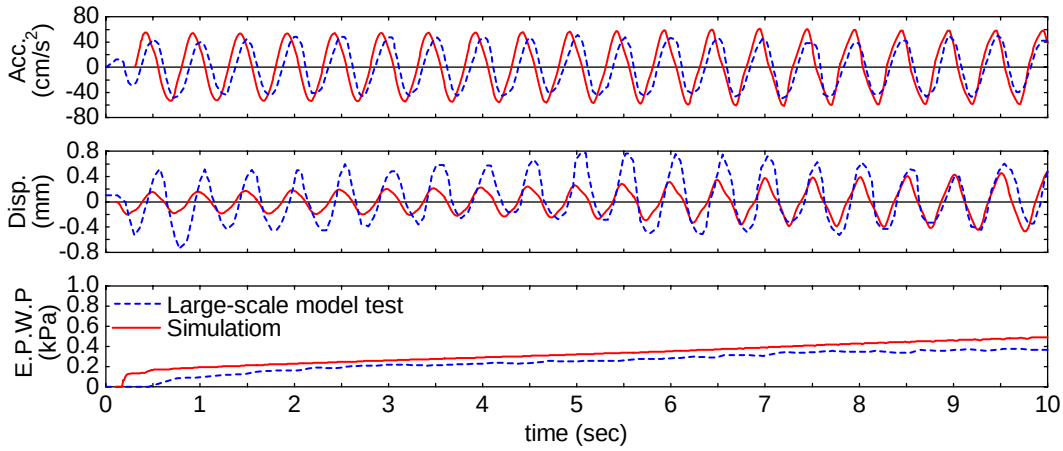

Figure 14 Results of analysis (R-502)

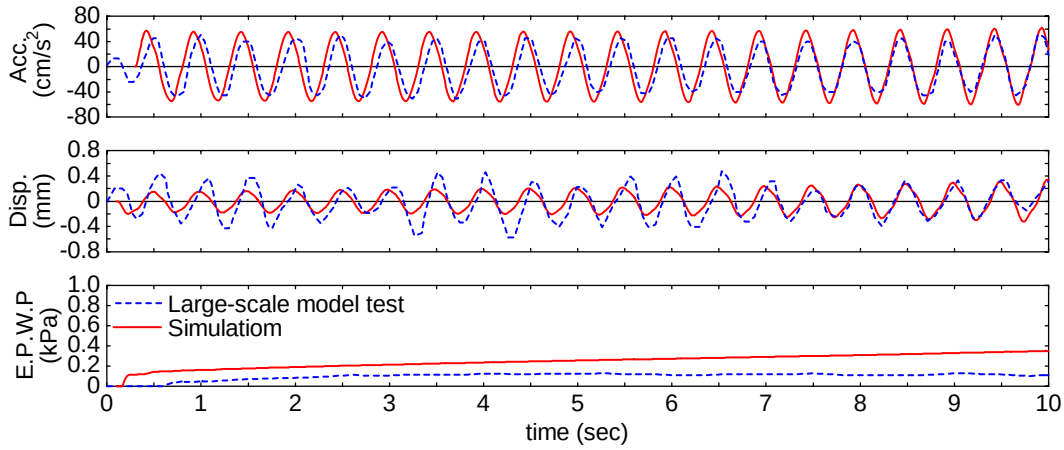

Figure 15 Results of analysis (R-702)

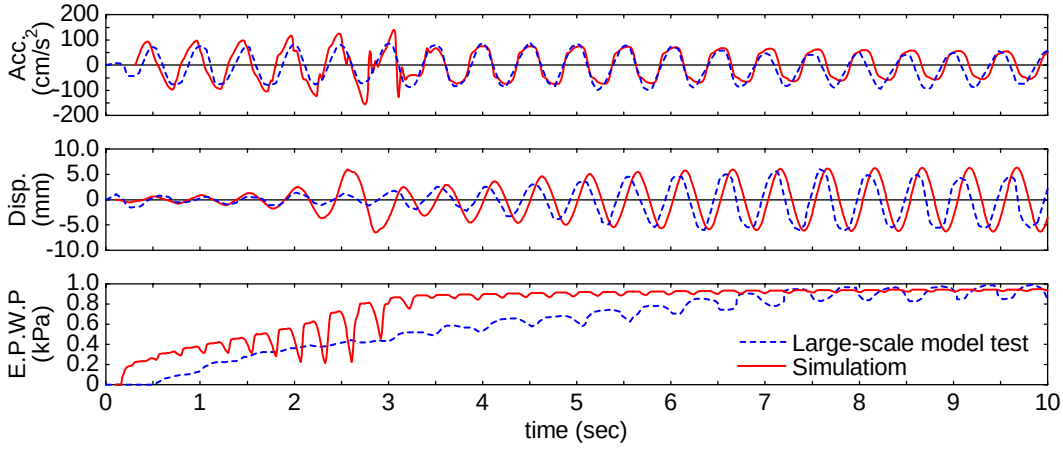

Figure 16 Results of analysis (R-703)

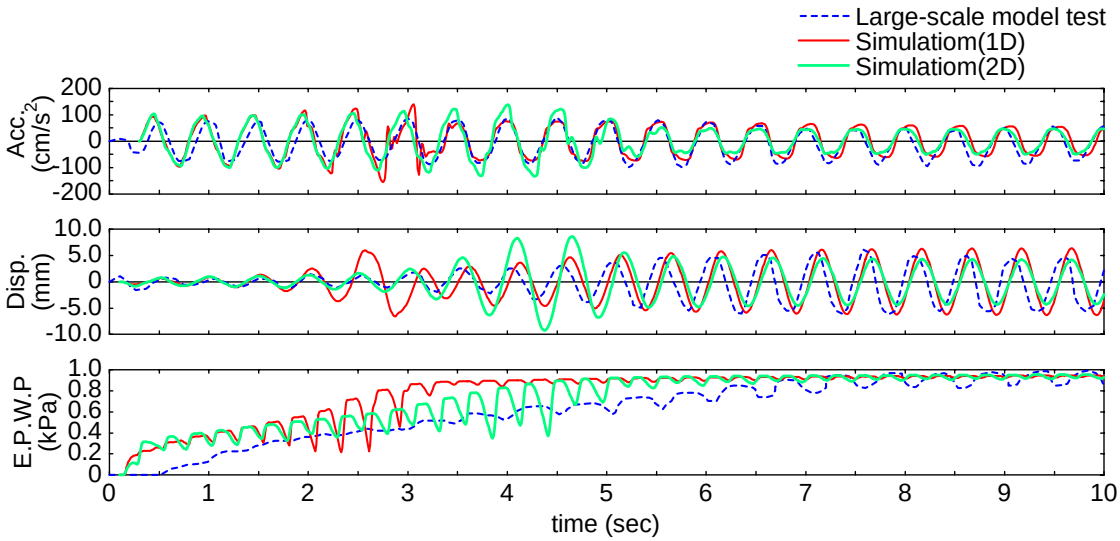

Figure 17 Results of 2D analysis (R-703)

CONCLUSIONS

A method to model the effect of drainage in high permeable ground is proposed to be used in dynamic effective stress analysis by undrained condition. The large-scale model tests were simulated in order to confirm the efficiency of the model. Responses, acceleration, displacement, excess pore water pressure, obtained by the simulation generally show good agreement with the test results. It is indicated that the generation of the excess pore water pressure is also restrained by considering the gravel rigidity in the model.

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REFERENCES

1. Seed H. B. and Booker J. R. “Stabilization of potentially liquefiable sand deposits using gravel drains.” Journal of the Geotechnical Engineering Division, ASCE, 1977; 103(GT7): 757-768.
2. Iai S., Matsunaga Y. and Kameoka T. “Strain space plasticity model for cyclic mobility” Report of the Port and Harbour Research Institute, 1990; 29(4): 27-56.
3. Towhata I, Ishihara K. “Modelling soil behaviour under principal stress axes rotation” Proceedings of the 5th International Conference on Numerical Methods in Geomechanics, Nagoya, 1985; 1: 523-530.
4. Ishihara K, Yoshida N, Tsujino S. “Modelling of stress-strain relations of soils in cyclic loading” Proceedings of the 5th International Conference on Numerical Methods in Geomechanics, Nagoya, 1985; 1: 373-380.
5. Iai S, Matsunaga Y, Kameoka T. “Parameter identification for a cyclic mobility model.” Report of the Port and Harbour Research Institute, 1990; 29(4): 57-83.

6. Ozutsumi O, Iai S. “Adjustment method of the hysteresis damping for multiple shear spring model” Proceedings of the 4th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, 2001.
7. Towhata I, Ishihara K. “Shear work and pore water pressure in undrained shear” Soils and Foundations, 1985; 25(3): 73-84.
8. Iai S. “Large scale model tests and analysis of gravel drains” Report of the Port and Harbour Research Institute, 1988; 27(3).
9. Ootaka M., Sato T., Ito T. and Nozu M. “Soil improvement effect of grain material in the pile as against liquefaction” Journal of Construction Management and Engineering, 2002; 700(VI-54): 183-188.