Chapter 25

LEAP-UCD-2017 Numerical Simulation at Meisosha Corp

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Abstract As a part of the LEAP-UCD-2017 project, type B simulations were performed to predict the results of nine selected centrifuge model tests using a two-dimensional effective stress analysis program incorporating the cocktail glass model. In Phase I of the numerical simulation, three parameter sets of the cocktail glass model were determined for three different dry densities. In Phase II, numerical models of the finite element method were made based on the data of the centrifuge model. At that time, for each centrifuge model test, we decided which parameter set of the cocktail glass model to apply by referring to the dry density of the analyzed ground. There was a tendency that the response values of the numerical simulation were slightly larger than the response values expected from the difference between the achieved dry density of the centrifuge model and the dry density on which the applied parameter set depends.

25.1 Introduction

It is considered advantageous that the constitutive model of sand for numerical simulation is based on the interaction between the sand particles. The strain space multiple mechanism model developed by Iai (Iai et al. 2011), which originated from the multi-inelastic spring model proposed by Towhata and Ishihara (1985), is such a model based on the soil particle interaction (Iai and Ueda 2016). The strain space multiple mechanism model has excellent extensibility. The model can be expanded to a three-dimensional constitutive model (Iai and Ozutsumi 2005). The model also can be expanded to a constitutive model for clay (Iai et al. 2015).

We participated as one of the numerical simulation teams of the LEAP-UCD-2017 project and got the opportunity to perform type B simulation targeted at the centrifuge model tests of liquefiable ground with a gently inclined surface. In the numerical simulations, we applied “the constitutive cocktail glass model” (Iai et al.

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2011) to reproduce the behavior of sand. The cocktail glass model is one of the achievements of the strain space multiple mechanism model.

The cocktail glass model was incorporated in a two-dimensional dynamic effective stress analysis program, “FLIP ROSE,” based on the finite element method. The program is able to analyze earthquake responses accompanied by liquefaction by considering the permeability of the ground. We used this program to perform type B LEAP simulation to confirm the performance of this constitutive model and program.

25.2 Constitutive Model

25.2.1 Model Description

The “cocktail glass model” was proposed for sandy soil based on the framework of a strain space multiple mechanism model for granular materials (Iai et al. 2011; Iai et al. 2013).

The macroscopic effective stress in granular materials is given by the average over contact forces $\mathbf{P}$ within a representative volume element with volume V as follows (Iai et al. 2013).

$$\boldsymbol{\sigma}' = \frac{1}{V} \sum l \mathbf{P} \otimes \mathbf{n} \quad (25.1)$$

See Fig. 25.1 for a definition of the variables.

The framework of the strain space multiple mechanism model is used to upscale the micromechanical structure into the macroscopic structure (Iai et al. 2013).

$$\boldsymbol{\sigma}' = -p\mathbf{I} + \int_0^\pi q \langle \mathbf{t} \otimes \mathbf{n} \rangle d\omega (\omega = 2\theta) \quad (25.2)$$

$$\langle \mathbf{t} \otimes \mathbf{n} \rangle = \begin{bmatrix} \cos \omega & \sin \omega \\ \sin \omega & -\cos \omega \end{bmatrix} \quad (25.3)$$

Here, p represents isotropic stress and q virtual simple shear stress (Iai et al. 2013). In order to define the relationship between macroscopic strain tensor $\boldsymbol{\varepsilon}$ and macroscopic effective stress tensor $\boldsymbol{\sigma}'$ through the structure defined by Eq. (25.2), let volumetric strain ε and virtual simple shear strain $\gamma(\omega)$ be defined as follows (Iai et al. 2013).

$$\varepsilon = \mathbf{I} : \boldsymbol{\varepsilon} \quad (25.4)$$

Fig. 25.1 Contact between two particles: normal $\mathbf{n}$, tangential direction $\mathbf{t}$, and contact force $\mathbf{P}$ (lai et al. 2013)

$$\gamma(\omega) = \langle \mathbf{t} \otimes \mathbf{n} \rangle : \boldsymbol{\varepsilon} \quad (25.5)$$

where the double dot symbol denotes a double contraction. To account for the effect of the volumetric strain due to dilatancy (ε_d), effective volumetric strain ε' is introduced as follows (lai 1993).

$$\varepsilon' = \varepsilon - \varepsilon_d \quad (25.6)$$

Effective volumetric strain ε' and virtual simple shear strain γ are used respectively to define isotropic stress p and virtual simple shear stress q as follows (lai et al. 2013).

$$p = p(\varepsilon') \quad (25.7)$$

Table 25.1 Main model parameters (based on Iai et al. 2011)

Symbol	Mechanism	Parameter designation
$K_{L/Ua}$	Volumetric	Bulk modulus corresponding to mean effective confining pressure p_a
r_K	Volumetric	Reduction factor of bulk modulus for liquefaction analysis
l_K	Volumetric	Power index of bulk modulus for liquefaction analysis
G_{ma}	Shear	Shear modulus corresponding to mean effective confining pressure p_a
ϕ_f	Shear	Internal friction angle
h_{max}	Shear	Upper bound for hysteretic damping factor
ϕ_p	Dilatancy	Phase transformation angle
r_{ε_d}	Dilatancy	Parameter controlling dilative and contractive components
$r_{\varepsilon_d^c}$	Dilatancy	Parameter controlling contractive component
q_1	Dilatancy	Parameter controlling initial phase of contractive component
q_2	Dilatancy	Parameter controlling final phase of contractive component
q_4	Dilatancy	Parameter controlling shear stiffness under liquefaction consideration
ε_d^{cm}	Dilatancy	Limit of contractive component
S_1	Dilatancy	Small positive number to avoid zero confining pressure
c_1	Dilatancy	Parameter controlling elastic range for contractive component
q_{us}	Dilatancy	Undrained shear strength (for steady-state analysis)

$$q = q(\gamma(\omega))$$

(25.8)

Dilatancy ε_d is decomposed into contractive dilatancy ε_d^c and dilative dilatancy ε_d^d as follows (Iai et al. 2011).

$$\varepsilon_d = \varepsilon_d^c + \varepsilon_d^d$$

(25.9)

The rate of ε_d^c is controlled by the irreversible (plastic) portion of virtual simple shear strain rate ($\dot{\gamma}_p$) and the rate of ε_d^d is controlled by virtual simple shear strain rate ($\dot{\gamma}$).

25.2.2 Model Parameters

Table 25.1 shows main model parameters for cocktail glass model.

25.3 Analysis Platform

25.3.1 Overview

We used a two-dimensional dynamic effective stress analysis program, “FLIP ROSE,” in the type B predictions. “FLIP ROSE” (Finite element analysis program

of Liquefaction Process/Response of Soil structure systems during Earthquakes) is based on the finite element method and infinitesimal strain formulation.

The program provides a multistage analysis function that enables the analysis to reproduce the initial stress state accurately by considering the construction process.

25.3.2 Elements

The program has 20 kinds of element. We used the pore water elements (partially drained) for modeling the behavior of the pore water and the cocktail glass model elements for modeling the behavior of the sand soil skeleton. The pore water element is based on u-p formulation by Zienkiewicz and Bettss (1982). The cocktail glass model element is based on the model proposed by Iai et al. (2011). For simulation of soil behavior below the ground water table, the pore water element (partially drained) is superposed with the cocktail glass model element by sharing the same nodes.

25.3.3 Governing Equations

Following Zienkiewicz and Bettss (1982), the governing equations for porous media saturated with pore water are given as a combination of equilibrium equation and mass balance equation of pore water as follows (u-p formulation).

$$\sigma_{ij,j} + \rho g_i = \rho \ddot{u}_i \text{ on } V_a, \quad (25.10)$$

$$(k_{ij} p_{,j})_{,i} - \dot{\varepsilon}_{ii} - (k_{ij} \rho_f g_j)_{,i} = -(k_{ij} \rho_f \ddot{u}_j)_{,i} + n \dot{p} / K_f \text{ on } V_b, \quad (25.11)$$

where

σ_{ij} : stress tensor, k_{ij} : coefficient of permeability, ε_{ij} : strain tensor,
 ρ : mass density (saturated density), ρ_f : mass density of pore water,
 g_i : gravity acceleration vector, u_i : displacement vector, n : porosity,
 K_f : bulk modulus of pore water.

25.3.4 Boundary Conditions

The following boundary conditions are assigned.

$$u_i = \bar{u}_i \text{ on } \Gamma_1, \quad (25.12)$$

Fig. 25.2 Boundaries for equilibrium equation and mass balance equation of pore water

$$\sigma_{ij}\nu_j = \bar{T}_i \text{ on } \Gamma_2, \quad (25.13)$$

$$p = \bar{p} \text{ on } \Gamma_3, \quad (25.14)$$

$$-\dot{w}_j\nu_j = k_{ij}(p_{,i} - \rho_f g_i + \rho_f \ddot{u}_i)\nu_j = \bar{q} \text{ on } \Gamma_4, \quad (25.15)$$

where

$\bar{u}_i$ —Displacement specified on boundary Γ_1 .

$\bar{T}_i$ —Traction specified on boundary Γ_2 .

$\bar{p}$ —Pore water pressure specified on Γ_3 .

$\bar{q}$ —Pore water inflow specified on Γ_4 .

$\dot{w}_j$ —Velocity of pore water relative to soil skeleton (average over total sectional area).

The union of the boundaries Γ_1 and Γ_2 is equal to the total area Γ of the analysis domain V_a , as shown in Fig. 25.2. The union of the boundaries Γ_3 and Γ_4 is equal to the total area Γ of the analysis domain V_b .

25.4 Model Calibration (Phase I)

25.4.1 Overview

The characterization tests, including cyclic undrained triaxial tests, of Ottawa F65 sand were conducted (El Ghoraiby et al. 2017, 2019). We performed the calibration of the model parameters for the constitutive model, that is, the cocktail glass model, as a part of the LEAP 2017 simulation exercise (Manzari et al. 2019). We are concerned here with the calibration procedures and the model parameters obtained. Also, we will show a comparison between the results of the numerical element simulations and the results of the corresponding cyclic undrained triaxial tests.

Table 25.2 Model parameter values determined through first step

e_0	ρ_d (kg/m ³)	ρ_{sat} (t/m ³)	n	G_{ma} (kPa)	p_a (kPa)	ν	$K_{L/Ua}$ (kPa)	ϕ_f (deg.)	ϕ_p (deg.)	h_{max}
0.515	1744.2	2.089	0.340	125,326	100	0.33	326,831	36.0	28.0	0.24
0.542	1712.6	2.070	0.351	119,147	100	0.33	310,716	36.0	28.0	0.24
0.585	1665.6	2.041	0.369	109,872	100	0.33	286,529	36.0	28.0	0.24

$p_a = 100$ kPa

25.4.2 Calibration Process (First Step)

We decided on the values of the parameters that can be determined without trial and error. Table 25.2 shows model parameter values determined through the first step. These parameter values were obtained through the following process:

1. Saturated mass density ρ_{sat} was obtained from void ratio e_0 , specific gravity ρ_s , and mass density of pore water ρ_w .
2. Porosity n was calculated from void ratio e_0 .
3. Mean effective confining pressure p_a corresponding to G_{ma} , $K_{L/Ua}$ was set to 100 kPa. This is an arbitrary value.
4. Initial shear stiffness G_{ma} for mean effective confining pressure p_a was calculated using the following equation (Hardin and Richart Jr 1963).

$$G_{ma} = 700 \frac{(2.17 - e_0)^2}{1 + e_0} (p_a)^{1/2} \quad (\text{kgf/cm}^2)$$

5. Poisson's ratio ν was assumed to be 0.33.
6. Bulk modulus $K_{L/Ua}$ was calculated by the relationship between bulk modulus, initial shear modulus, and Poisson's ratio for isotropic elastic medium.
7. Using the following equation, internal friction angle ϕ_f was calculated from the gradient M measured on the compression side of the effective stress path diagram.

$$M = 6 \sin \phi_f / (3 - \sin \phi_f)$$

8. Phase transformation angle ϕ_p was assumed to be 28 degrees.
9. Upper bound for hysteretic damping factor h_{max} was assumed to be 0.24.

25.4.3 Calibration Process (Second Step)

We decided on the values of the parameters that should be determined by trial and error. During the trial and error procedure, we kept the values determined through the first step. Table 25.3 shows the model parameter values determined through the

Table 25.3 Model parameter values determined through second step

e_0	ρ_d (kg/m ³)	r_K	l_K	r_{e_d}	$r_{e_d}^c$	q_1	q_2	q_4	ε_d^{cm}	S_1	c_1	q_{us} (kPa)
0.515	1744.2	0.225	1.4	0.444	0.20	20	0.9	1.0	0.0366	0.005	2.7	0
0.542	1712.6	0.3	1.4	0.333	0.95	20	1.3	1.0	0.0561	0.005	2.1	0
0.585	1665.6	0.3	1.4	0.333	1.70	20	1.3	1.0	0.0599	0.005	1.5	0

Fig. 25.3 Procedure of element simulation for performing calibration

second step. In order to obtain these parameters, element simulations under plane strain condition were carried out by the procedure shown in Fig. 25.3.

According to the characterization test results, the axial strain time histories were unidirectional, but this tendency was not seen in the element simulations. Therefore, we did not select the liquefaction resistance curves determined with 2.5% single amplitude axial strain, shown in the report (El Ghoraiby et al. 2017, 2019), as targets of the calibration. Instead, we adjusted the parameters by fitting mainly axial strain double amplitude shown in the analytical time histories to those obtained from the triaxial tests.

25.4.4 Results of the Simulation of the Cyclic Triaxial Tests

Figure 25.4 shows comparisons of the liquefaction resistance curves between the test results and the corresponding element simulation results. Element simulations were carried out using the parameters shown in Tables 25.2 and 25.3. The liquefaction resistances plotted in Fig. 25.4 were determined with 2.5% single amplitude axial strain. Figure 25.5 shows sample comparisons of the effective stress path, the deviatoric stress–axial strain relation, the mean effective stress–axial strain relation and the time history of the excess pore water pressure ratio. Figure 25.6 shows comparisons of the time histories of mean effective stress and axial strain. In

Fig. 25.4 Liquefaction strength curves obtained from cyclic stress-controlled triaxial tests (El Ghoraihy et al. 2017, 2019) and FLIP ROSE element simulations (Filled markers correspond to test results. Open markers correspond to simulation results.)

Fig. 25.5 A sample comparison of the simulated versus measured stress-strain responses and effective stress path ($e_0 = 0.585$, $\rho_d = 1665.6 \text{ kg/m}^3$; $\text{CSR} = 0.14$)

Fig. 25.6 A sample comparison of the simulated versus measured mean effective stress and axial strain histories ($e_0 = 0.585$, $\rho_d = 1665.6 \text{ kg/m}^3$; $\text{CSR} = 0.14$)

Figs. 25.5 and 25.6, the responses in the case of $e_0 = 0.585$, $\rho_d = 1666 \text{ kg/m}^3$ and $\text{CSR} = 0.14$ are shown as an example.

25.5 Type B Simulation (Phase II)

25.5.1 Overview

Type B simulations were carried out to predict the response of nine cases of centrifuge model tests conducted in seven organizations in the LEAP 2017 project. The centrifuge model was a ground made of Ottawa F65 sand whose surface was gently inclined. The ground dimensions were 20 m in the shaking direction and 4 m average depth (prototype scale). Numerical simulations were carried out using 1 g field equivalent analysis model. The input ground motion used in the analysis was an acceleration time history with maximum amplitude of about 0.15 g and duration of about 20 s. The code names of the nine centrifuge model tests were CU2, Ehime2, Kaist1, Kaist2, KyU3, NCU3, UCD1, UCD3, and ZJU2. See “LEAP 2017 Simulation Exercise Phase II: Type B Prediction of Centrifuge Experiments Guidelines for submission of the numerical simulations.” for details (George Washington University et al. 2017).

25.5.2 Model Geometry/Mesh and Boundary Conditions

Figure 25.7 shows details of the finite element model. We divided the whole area into four sub-areas so as to be able to assign the different material parameters to each

Fig. 25.7 Finite element model: soil structure

Fig. 25.8 Finite element model: mesh and boundary conditions

area based on the soil density. However, we set the same material parameters to each area as discussed later. Figure 25.8 shows the finite element mesh and the boundary conditions.

Each element shown in Fig. 25.8 is a cocktail glass model element. Every cocktail glass model element is accompanied by a pore water element (partially drained) by sharing the same nodal points. Total number of the cocktail glass model elements was 2686; hence, total number of the pore water elements was also 2686. Total number of the nodal points was 2800.

The boundary conditions we applied are shown in Fig. 25.8.

25.5.3 Analysis Conditions

1. Time integration method.

The coupled set of equations consisting of equilibrium equation and mass balance equation of pore water was integrated in time by using generalized time integration method (SSpj method, Zienkiewicz et al. (2005)). We set the time integration interval Δt to 0.001 sec so as to avoid the numerical instability in all cases. We also performed numerical analyses for 50 s, therefore, the number of time steps was 50,000 in each case.

2. Rayleigh damping.

Rayleigh damping was used instead of the intrinsic viscous damping of the soil (Tatsuoka et al. 2002). We applied Rayleigh damping shown in Eq. (25.16) to the equilibrium equation.

$$\mathbf{C} = \alpha \mathbf{M} + \beta \mathbf{K} \quad (25.16)$$

where $\mathbf{C}$ is a Rayleigh damping matrix, $\mathbf{M}$ is a mass matrix. $\mathbf{K}$ is an initial tangential stiffness matrix at the beginning of dynamic analysis and is constant throughout the entire analysis time. We set parameters α, β as follows in all cases.

$$\alpha = 0.0, \beta = 1.555 \times 10^{-4} \quad (25.17)$$

We performed eigenmode analysis under undrained condition in each centrifuge experimental case. The frequency of the first eigenmode was almost the same throughout the entire cases. They were approximately 10.24 Hz. Coefficient β in Eq. (25.16) was set to 1.555×10^{-4} so that the damping constant h of the first eigenmode becomes 0.005.

25.5.4 *Material Properties and Constitutive Model Parameters*

Model parameters determined in Phase I were used as they were, except for mass density, porosity and permeability coefficient.

According to the following procedure, we calculated the dry density (ρ_d) of the ground for each case of the centrifuge model test.

1. We read the cone penetration resistance (q_c) at three or four depths (1.5 m, 2.0 m, 2.5 m, 3.0 m) from the graph representing the depth dependence of q_c submitted by each institution conducting centrifuge model test.
2. The dry density (ρ_d) at each of these depths was read from the correlation diagram (Manzari 2017) of the cone penetration resistance (q_c) to the dry density (ρ_d).
3. The dry densities obtained at these depths were averaged.

Table 25.4 shows mean dry density and coefficient of variation of dry densities for each centrifuge model experiment. As shown in Table 25.4, the coefficients of variation are mainly smaller than or equal to 1.0%; therefore, we decided to apply these mean values to the entire soil layer in each centrifuge model test.

In the Phase I, the parameter sets of the cocktail glass model were determined for three cases of dry densities; $\rho_d = 1665.6, 1712.6, 1744.2 \text{ kg/m}^3$. Each centrifuge test case had to be assigned with one of these parameter sets having the nearest dry density, because the liquefaction characteristics of soil with any other dry density except those of the three cases in Phase I was unknown.

Table 25.4 Mass density, porosity, and permeability coefficient

Centrifuge test	Dry density		Dry density (CV)	Void ratio e_0	Saturated mass density ρ_{sat} (t/m ³)	Porosity N	Permeability, $T = 20\text{ }^{\circ}\text{C}$	
	(Mean) ρ_d (kg/m ³)						k (m/s)	μ^*/g^* (cSt)
CU2	1581		0.010	0.677	1.984	0.404	1.108E-4	1.175
Ehime2	1666		0.007	0.590	2.037	0.371	1.020E-4	1.0
KAIST1	1681		0.006	0.577	2.047	0.366	1.005E-4	0.897
KAIST2	1591		0.007	0.666	1.990	0.400	1.098E-4	0.936
KyU3	1652		0.012	0.604	2.028	0.377	1.034E-4	0.991
NCU3	1651		0.010	0.606	2.028	0.377	1.035E-4	1.0
UCD1	1659		0.005	0.597	2.033	0.374	1.027E-4	1.0
UCD3	1626		0.007	0.630	2.012	0.386	1.060E-4	1.0
ZJU2	1585		0.009	0.671	1.987	0.402	1.103E-4	1.0

Note 1: Dry density (Mean) is the average value of ρ_d based on the penetration resistance q_c at 4 or 3 depths (1.5 m, 2.0 m, 2.5 m, 3.0 m) obtained by the cone penetration test (CPT) during the centrifugal experiment

Note 2: Dry density (CV) represents coefficient of variation of $e_0 = \rho_d/\rho_d - 1$

Note 3: Void ratio (e_0) was calculated using the equation of $e_0 = \rho_d/\rho_d - 1$

Note 4: Saturated density (ρ_{sat}) was calculated using the equation of $\rho_{\text{sat}} = (\rho_s + e_0)/(1 + e_0\rho_w)$

Note 5: Porosity (n) was calculated using the equation of $n = e_0/(1 + e_0)$

Note 6: Coefficient of permeability at $T = 20\text{ }^{\circ}\text{C}$ (k) was set as a value depending on the void ratio e_0 with reference to El Ghorabiy et al. (2017)

Note 7: Adjusted coefficient of permeability at $T = 20\text{ }^{\circ}\text{C}$ (k^*) was calculated using the equation of $k^* = k/(\mu^*/g^*)$

Note 8: $\rho_s(=2.65\text{ t/m}^3)$ is mass density of soil particle, $\rho_w(=1.0\text{ t/m}^3)$ is mass density of pore water

Fig. 25.9 Dry densities in the centrifuge model tests and in Phase I

Fig. 25.10 Initial state of the model for UCD1—vertical displacement (m) due to gravity

As shown in Fig. 25.9, the dry density of each centrifuge test was closest to the dry density of the loosest case in Phase I. Therefore, in the type B simulation, we applied a parameter set of the loosest specimen, whose ρ_d was 1665.6 kg/m³, to any case in the centrifuge model test. This implies that the liquefaction strength in all centrifuge model test cases becomes the same with each other, and then the results of the seismic response analyses in the type B simulations may become similar.

As shown in Table 25.4, saturated mass density (ρ_{sat}), porosity (n) and coefficient of permeability at $T = 20\text{ }^{\circ}\text{C}$ (k^*) were determined for each centrifugal test case based on their mean value of mass density ρ_d .

25.5.5 *State of Stresses and Internal Variables of the Constitutive Model in the Pre-shaking Stage*

Figures 25.10, 25.11, 25.12, 25.13, 25.14, and 25.15 show the contour plots obtained by initial self-weight analysis for the model of UCD1. Since the plot diagrams of the other cases are almost the same as the diagrams for UCD1, these drawings of the other cases are omitted. In these figures, compression of stress is represented by negative sign, and tension is represented by positive sign.

Figures 25.10 and 25.11 show the distributions of vertical displacement and horizontal displacement, respectively. Figure 25.12 shows the distribution of pore

Fig. 25.11 Initial state of the model for UCD1—horizontal displacement (m) due to gravity

Fig. 25.12 Initial state of the model for UCD1—pore water pressure due to gravity

Fig. 25.13 Initial state of the model for UCD1—vertical effective stress σ_{yy}' (kPa) due to gravity

Fig. 25.14 Initial state of the model for UCD1—horizontal effective stress σ_{xx}' (kPa) due to gravity

Fig. 25.15 Initial state of the model for UCD1—shear stress τ_{xy} (kPa) due to gravity

water pressure. According to this figure, if the depth from the assumed water surface (see Fig. 25.8) is the same, the pore water pressure will be the same. The effective vertical stress distribution shown in Fig. 25.13 is a bedded distribution according to the depth. Figures 25.14 and 25.15 show the distributions of effective horizontal stress and shear stress, respectively.

25.5.6 Results of the Dynamic Analysis

Seismic response analysis of 50 s was conducted for each case. The excitation was carried out in the first half for 20 s, and in the latter half for 30 s the drainage of the increased excess pore water pressure was tracked. Here, we show comparisons of time history between the experiment results and numerical simulation results.

Figure 25.16 shows the points for which the comparisons of time history charts between experiment results and numerical simulation results were made. Figure 25.17 shows the comparisons of horizontal acceleration time histories at point AH4. Figure 25.18 shows the comparisons of horizontal displacement time histories at point Disp_0. Figure 25.19 shows the residual horizontal displacement distribution of the case “UCD3” obtained by numerical simulation. Figure 25.20 shows the comparisons of excess pore water pressure time histories at point P1. Figure 25.21 shows the excess pore water pressure distribution at time 15.1 s obtained by numerical simulation for the case “UCD3.”

Fig. 25.16 Output points, added to “Baseline schematic for LEAP 2017 centrifuge experiment.”, for time histories: AH4 (Horizontal Acc.), Disp_0 (Horizontal Disp.), P1 (E. P. W. P.)

Fig. 25.17 Acceleration time histories at AH4; center of the model, 0.5 m below the ground surface

Figure 25.17 shows that the acceleration time histories by numerical simulations trace the results of experiments well, except spikes, which can be seen in the results of numerical simulations. These spikes seem to indicate that there is attenuation that is not taken into consideration in the analysis. For example, attenuation due to sliding friction between sand and rigid wall can be mentioned. In any case, if there is attenuation that is not considered in the analysis, the analysis result will have a larger displacement.

Figure 25.18 shows that the horizontal displacement time histories by numerical simulations trace well the results of experiments in case of Ehime2, KAIST2, KyU3, and UCD3 except oscillations, which can be seen in the simulation results. The cause of these oscillations seems to be the same as the cause of the spikes in the acceleration time histories. If we take into account the attenuation, then the amplitude of the oscillations will decrease.

On the other hand, according to Fig. 25.9, the results of the numerical simulation may be underestimated for cases of “CU2,” “KAIST2,” “UCD3” and “ZJU2,” and expected to be correct in other cases. Because, the dry densities of these four cases are below the dry density of the loosest sand in Phase I and the dry densities of the other cases are roughly equal to the dry density of the loosest sand in Phase I.

Fig. 25.18 Horizontal displacement time histories at Disp_0; center of the model, on the surface

Fig. 25.19 Final state of the model for UCD3 at 50 s—horizontal displacement

If we take into account the unconsidered damping in the numerical analyses, then the residual displacements in Fig. 25.18 will decrease and the tendency of these residual displacements appears to become close to the above expectation based on Fig. 25.9.

According to the comparison of the time history of the excess pore water pressure shown in Fig. 25.20, the results of numerical simulation generally correlate with the test results except for the water pressure dissipation phase. However, in terms of maximum value of excess pore water pressure, the numerical simulation results in many cases exceed the test results.

Fig. 25.20 Excess pore water pressure time histories at P1; center of the model, depth 4 m

Fig. 25.21 State of the model for UCD3 at 15.1 s—Excess pore water pressure distribution

25.6 Conclusion

As a part of the LEAP-UCD-2017 project, type B simulations were performed for centrifuge model tests using a two-dimensional effective stress analysis program “FLIP ROSE” incorporating a cocktail glass model (Iai et al. 2011). The centrifuge model tests were conducted as a part of the LEAP-UCD-2017 project to investigate the behavior of the liquefiable ground with gentle slope.

In Phase I of the numerical simulation, three parameter sets of the cocktail glass model were determined by trial and error so as to reproduce the results of the

undrained cyclic triaxial tests for the specimens of three different dry densities. In Phase II, models were made based on the data of the centrifuge model tests. For each centrifuge model test, we decided which parameter set of the cocktail glass model to apply by referring to the dry density of the ground. As a result, for all of the centrifuge model tests, the parameter set obtained based on the triaxial test result of the specimen with the loosest dry density was applied. It seems that it is difficult to adjust the dry density of the centrifuge model and in many of the models the density was lower than that of the loosest specimen.

Since it was a type B simulation, numerical simulations were performed without knowing the experiment results. Acceleration time histories, displacement time histories and excess pore water pressure time histories simulation results were generally consistent with the results of the centrifugal model tests. There was a tendency that the response values of the numerical simulation were slightly larger than the response values expected from the difference between the dry density of the ground of the centrifuge model and the dry density of the specimen on which the applied parameter set depends. These differences seem to be related to the attenuation due to sliding friction between the ground and the rigid wall, which is not taken into account in the numerical simulation.

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