Numerical simulation of centrifuge tests to evaluate the performance of desaturation by air injection on liquefiable foundation soil of light structures

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Abstract

Four dynamic centrifuge tests were conducted to study the performance of a newly developed countermeasure technique against soil liquefaction: “desaturation by air injection”. In the experiment, liquefiable foundation soils below lightweight structures were desaturated by the air injection technique and base shaking was imparted to the models to obtain a comprehensive set of response data. In this study, numerical simulations of these experiments were conducted by using a two-phase (solid and fluid) fully coupled finite-element code, Coupled Analysis of Liquefaction (LIQCA-2D), to validate the numerical procedures. The mechanical properties of the soil in the saturated and desaturated zones in the models were exactly the same, with the exception of the degree of saturation. The simulation attempted to examine the desaturated models by changing the compressibility of the pore fluid, in which all input parameters for saturated and desaturated models were the same except for the bulk modulus, $K_f$. Numerical results were comparable with the test results in terms of excess pore pressures and settlement of structures for both saturated and desaturated models. This validates the numerical procedure and further assures the effectiveness of desaturation by the air injection technique as a liquefaction countermeasure.

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Keywords: Centrifuge test; Numerical analysis; Liquefaction countermeasure; Degree of saturation; Shallow foundation

1. Introduction

Shallow foundations of residential buildings on liquefiable soil layers have often been damaged during large earthquakes. In recent earthquakes, such as the 2011 Darfield earthquake in New Zealand and the 2011 off the Pacific coast of Tohoku Earthquake, massive destruction of residential buildings occurred, which urgently necessitates reliable and cost-effective countermeasure techniques to remediate liquefiable foundation soils of existing residential houses (The Japanese Geotechnical Society, 2011; Yasuda and Ishikawa, 2012; Orense, 2011; Cubrinovski et al., 2012; Nakai and Sekiguch, 2012). To reduce such damage to existing residential houses, a limited number of remediation methods are available in practice. These methods are primarily based on densification, solidification, and replacement techniques and are excessively expensive.

Because the degree of saturation has a significant effect on the liquefaction resistance of soils, methods of soil desaturation have been studied as remedial measures for liquefaction, which include water electrolysis (Yegian et al., 2006, 2007) and gas...
In recent years, an innovative liquefaction countermeasure technique desaturation by air injection has been developed (Okamura et al., 2011, 2012a, 2012b) and has attracted significant interest from engineers because of its extreme affordability and environmental friendliness. It has been reported that injection of air into the ground can substantially lower the degree of saturation of the subsoil (Tokimatsu et al., 1990; Okamura et al., 2003) and this unsaturated condition of the desaturated soils lasts for an extensive time period, typically decades or more (Okamura et al., 2006). Okamura et al. (2011) conducted an in situ air injection test and confirmed that soil in the zone of influence, which is approximately 3.5 m from the injection port, was effectively desaturated. Tomida (2014) conducted a similar test under a road embankment with relatively high air injection pressure and found that the radius of the desaturated zone from the single injector, which increased with increasing air pressure and injection time, extended 9 m in 18 h. Because the material for this technique is free, drilling and installation of injection pipes amounts to the majority of execution costs. A dramatic reduction in estimated execution costs has been achieved by increasing the radius of the desaturated zone.

The effect of degree of saturation on the liquefaction resistance of soils has been studied since the 1960s through undrained cyclic shear tests. The existence of air in pores of soils reduces the bulk modulus of pore fluid (that is, the air–water mixture), which results in increased liquefaction resistance. Changes in the volume of pore fluid during cyclic shearing was found to be the factor dominating this mechanism of enhancing soil resistance to liquefaction (Okamura and Soga, 2006; Unno et al., 2008).

Regarding numerical simulation, limited research has been conducted on this topic. Mitsuji (2008) tried to simulate seismic behavior of unsaturated sand deposits with a one-dimensional effective stress analysis, in which incomplete saturation of the sand was modelled by reducing the bulk modulus of pore fluid. He found that velocity, displacement, and shearing strain of the ground decreased with decreasing the bulk modulus. Gao et al. (2013) developed a computational model based on the Biot's two phase mixture theory and conducted numerical simulations on the behaviors of unsaturated soils under cyclic loadings. They also studied effects of the bulk modulus of the pore fluid on the pore pressure evolution at different initial degree of saturation. Similarly, Yashima et al. (1995) conducted three dimensional liquefaction analysis based on the Biot's theory to observe effects of pore fluid compressibility due to the imperfect saturation of reclaimed soil layers. They found that the strong motion array records of the Port Island observed during the 1995 Hyogoken-Nambu Earthquake were simulated reasonably well. All these numerical analysis mentioned above have dealt with the pore pressure and the acceleration responses of level ground.

Recently, highly instrumented centrifuge tests were conducted to assess the performance of the desaturation technique as a liquefaction countermeasure for soils immediately beneath existing structures (Marasini and Okamura, 2015). The recorded model responses provide an unique opportunity to verify performance of the numerical procedures. In this study an attempt was made to verify the numerical procedure to simulate deformation of locally desaturated soil with structures. This paper presents the results of a computational study based on a comprehensive experimental set of data. The four centrifuge models tested using the geotechnical centrifuge at Ehime University were simulated by using Coupled Analysis of Liquefaction (LIQCA-2D) (Oka et al., 1994, 1999), a finite-element method (FEM)-based effective stress analysis.

2. Overview of centrifuge tests

Marasini and Okamura (2015) performed four centrifuge tests to investigate the effectiveness of desaturation by air injection to mitigate liquefaction-induced settlement of lightweight structures. Schematic illustrations and test conditions of the centrifuge models are presented in Fig. 1 and Table 1, respectively. Models of a structure, with a base contact pressure of either 10 kPa or 35 kPa, resting on either fully saturated or desaturated foundation soils, were prepared at 1:50 scale and tested in the centrifuge at 50g.

The models were prepared in a rigid container with internal dimensions of a 430 mm length, 120 mm width, and 230 mm depth. The soil used for the foundation soil layer was Toyoura sand. A 20-mm-deep dense sand layer was prepared at a relative density of \( D_r = 90\% \) on the base of the container. For Models M1-2 and M2-2, a two-dimensional air injector with 1-mm-wide orifices on both sides was placed on the dense sand layer. The dry sand was then poured into the container to form a 120-mm-deep uniform sand deposit at \( D_r = 50\% \). The models were fully saturated in a vacuum environment with an aid of the CO2 replacement technique. The substituted pore fluid was deaired viscous fluid, with a viscosity 50 times that of water (equivalent to g level).

On completion of the saturation process, the degree of saturation (\( S_r \)) of the model was measured by the method developed by Okamura and Inoue (2012). The estimated degree of saturation of all models was very high, in a range between 99.70% and 99.84%.

A mild steel plate, 120 mm wide and either 2.5 mm or 9 mm high, representing the two-dimensional shallow foundation of residential houses, was placed on the surface with two potentiometers for measurement of vertical settlement. The base contact pressures of the lighter and heavier foundations at 50 g were 10 kPa and 35 kPa, respectively, which were equivalent to one- to two-story residential buildings. Each model was then set on the geotechnical centrifuge at Ehime University (http://www.cee.ehime-u.ac.jp/~gm/indexE.html), and centrifugal acceleration was gradually increased to the target level of 50g. The acceleration was kept constant to allow ample time for excess pore fluid to drain through the stand pipes until the height of the water table reached 40 mm below the ground surface.

For Models M1-2 and M2-2, air was injected at 50 g through the injector set on the dense sand layer for simulating in situ air injection to desaturate soil just below the structures. The air
injection pressure was increased slowly. When injection air pressure reached \( P_{\text{inj}} = P_{\text{hyd}} + \text{AEV} \), where \( P_{\text{hyd}} \) denotes the hydrostatic pressure at the depth of the injection and AEV denotes the air entry value of sand, air started to flow and the water level rose accordingly in the model and raised water level was measured by the pore pressure sensor installed in the model. The air injection was continued for ample of time and halted when air bubbles noticed on the top of soil layer. After the air injection was halted, the increased pore pressure slowly came down and reached constant with higher than that before the air injection. The difference in water levels (recorded in pore pressure sensor) before and after air injection corresponded to the volume of air remaining in the soil influenced zone. Sand in the zone of influence of air injection changed in color and was detected through both video images in-flight and visual observation after the tests, as shown in Figs. 13 and 14. Total pore volume of influenced zone was estimated by marking the color changed on the model after the shaking was over. From which volume of residual air on the influenced zone can be easily estimated. The degrees of saturation in the desaturated zone following air injection were estimated to be 87% for Model M1-2 and 85% for Model M2-2.

On completion of the drainage of excess pore fluid, one-dimensional horizontal shaking was imparted along the long axis of the model by using a mechanical shaker simulating a sinusoidal wave with a dominant frequency of 0.8 Hz and acceleration amplitude of 1.9 m/s².

3. Pore fluid bulk modulus and its pressure-level dependency

The existence of air in the pore of a soil is considered to enhance the liquefaction resistance of the soil in two ways. The

![Diagram](image-url)
first mechanism is that air in pores absorbs the generated excess pore pressure by reducing its volume. The bulk modulus of the pore fluid degrades significantly by the presence of air bubbles. The contraction of pore fluid (that is, air–water mixtures) dominates this mechanism. The second is the matric suction of unsaturated soils, which increases the effective stresses and thus the strength of soil mass (Bishop and Blight, 1963). For most liquefiable soils, however, matric suction is less significant than the effective stress of soils at the depth of practical concern (Okamura and Soga, 2006). For unsaturated soils with a degree of saturation higher than 80%, air bubbles exist in the occluded form within the pore fluid; the diameters of these bubbles are generally on the same order of grain size (Fredlund and Rahardjo, 1993). Under these conditions for the particular sand used in these tests, pore air and pore water pressures are considered to be equal and the effect of matric suction is neglected in this analysis (Okamura and Noguchi, 2009).

For a small change in pore pressure, $\Delta p$, the volumetric strains of air and pore water are

$$
\epsilon_a = \frac{\Delta p}{K_a}
$$

(1)

$$
\epsilon_w = \frac{\Delta p}{K_w}
$$

(2)

Eq. (1) can be rewritten by using Boyle’s law as follows:

$$
\epsilon_a = \frac{\Delta p}{P_0 + \Delta p}
$$

(3)

and the volumetric strain of the fluid (water–air mixture), $\epsilon_{vf}$, is

$$
\epsilon_{vf} = \frac{\Delta p}{K_f} = \left[(1 - S_r)\epsilon_a + S_r\epsilon_w \right] = \Delta p \left( \frac{1 - S_r}{K_a} + \frac{S_r}{K_w} \right)
$$

(4)

where $S_r$ is the degree of saturation of the soil mass; $P_0$ is the absolute hydrostatic pressure; and $K_a$, $K_w$, and $K_f$ are the bulk moduli of air, water, and fluid, respectively. The volumetric strain and bulk modulus of the fluid attain their highest values when $\Delta p$ attains its maximum possible value during an earthquake, which is equal to the initial effective vertical stress $\sigma_{vo}$. The maximum bulk modulus of the fluid, designated hereafter as the potential bulk modulus, is

$$
K_f' = \frac{(P_0 + \Delta p)K_w}{K_w(1 - S_r) + (P_0 + \Delta p)S_r} \leq \frac{(P_0 + \sigma_{vo})K_w}{K_w(1 - S_r) + (P_0 + \sigma_{vo})S_r}
$$

(5)

Air dissolution into water is not taken into account

Fig. 2 shows the variations in the potential bulk modulus of pore fluid at a depth of 2 m below the groundwater table (corresponding to location C1 in the model) with degrees of saturation in Models M1-2 and M2-2. The potential bulk modulus decreases dramatically with a small reduction from 100% in the degree of saturation. Similarly, Fig. 3(a) and (b) depicts the maximum (possible) bulk modulus and volumetric strain on the center line of desaturated centrifuge models M1-2 and M2-2. This figure indicates significant dependency of the stress level on the potential bulk modulus and potential volumetric strain, whereas for the fully saturated models (M1-1 and M2-1), the bulk modulus of pore fluid remains the same as that of water, $2 \times 10^6$ kPa, irrespective of the stress level.

4. Numerical method

Oka et al. (1999) proposed a cyclic elastoplastic model based on the nonlinear kinematic hardening rule to conduct a study on the liquefaction behavior of saturated sandy soils during dynamic loading. In the present study, the effective-stress-based numerical code LIQCA-2D, developed by Oka et al. (1994, 1999), is used to simulate the centrifuge models.

In this numerical method, the governing equations for the coupling problems between the soil skeleton and pore water were obtained with Biot’s two-phase mixture theory (Biot, 1962). For the dynamic analysis, the displacement–pore pressure ($u$–$p$) formulation method was used. Similarly, to discretize Biot’s governing equations for a two-phase mixture, the FEM is adopted with the virtual work theorem. In this method, however, the FEM was used for the spatial discretization of the equilibrium equation, whereas the finite-difference method (FDM) was used for spatial discretization of the pore water pressure in the continuity equation (Akai and Tamura, 1978). Stresses and strains were defined at the center of an element by using the reduced numerical integration method, which can avoid shear locking under undrained conditions. Although the details of this method are provided in Oka et al. (1994), the basic assumptions to formulate the governing equations are as follows:

1. Infinitesimal strain is used;
2. Relative acceleration of the fluid phase to that of the solid phase is negligible; and
3. Grain particles in the soil are incompressible.

The derived equilibrium equation for the mixture is as follows:

$$
\rho \ddot{u}_i = \sigma_{ij;j} + \rho b_i
$$

(6)

where $\rho$ is the overall density, $\dot{u}_i$ is the acceleration of the solid, $\sigma_{ij}$ is the total tensor, and $b_i$ is the body force.

The continuity equation is as follows:

$$
\rho \dot{\varepsilon}_{ij}^{\text{solid}} - p_{ij} - \frac{\gamma}{k} \dot{\varepsilon}_{ij}^{\text{fluid}} + \frac{\mu r}{k} \dot{p} = 0
$$

(7)

At C1 in the centrifuge model
where $\rho$ is the density of pore fluid, $\mu_s$ is the spatial derivative of the acceleration of the solid, $p$ is the pore water pressure, $\gamma_w$ is the unit weight of the fluid, $k$ is the coefficient of permeability, $\varepsilon_s$ is the volumetric strain rate of the solid, $n$ is porosity, $K_f$ is the bulk modulus of the fluid and ($\cdot$) denotes the time differentiation.

In this numerical model, compressibility of the sand is expected to be modelled properly by reducing the bulk modulus of pore. But the variation in the permeability of sand with degree of saturation ($S_r$) is not properly considered in the simulation. The sand in the desaturated zone with $S_r$ 85% has significantly lower permeability than that in the saturated zone. This limitation of the simulation may overestimate the power water migration and pore pressure reduction in the neighborhood. Similarly, the effects of suction of sand in the desaturated zone on liquefaction resistance is also not considered in the simulation model. But the suction in the unsaturated soil with air bubbles in occluded form is low. Suction of the sand at degree of saturation of 85% (estimated $S_r$ in the centrifuge models) on the imbibition path of the SWRCC (Soil water retention characteristic curve) is less than 1 kPa, and an increase in the liquefaction resistance due to this small suction can be negligible (Okamuras and Noguchi, 2009). Similarly, dissolution of air in the pore due to pore pressure generation is occurred during the earthquake shaking, the effects of this air dissolution is also not considered in the simulation.

5. Determination of input parameters

The determined input parameters are summarized in Table 2 for both loose ($D_r = 50\%$) and dense ($D_r = 90\%$) Toyoura sand used in the centrifuge models. Parameters including $\rho$, $e_0$, $k$, $\lambda$, $\kappa$, $M_f$, and $M_m$ were taken from the test results conducted by Fukushima and Tatsuoka (1984). The quasi-overconsolidation ratio ($OCR^*$) was established at 1.0 based on the experimental conditions. The initial shear modulus ratio, $G_0/\sigma'_m$, was established according to Oka et al. (2004). Thereafter, the remaining input parameters were determined by the data adjustment method to reproduce the results of the undrained cyclic triaxial test (Toki et al., 1986; Yamamoto et al., 2009) in terms of the liquefaction strength curve, shear stress–strain relation, and effective stress path.

Fig. 4 shows the test results (Toki et al., 1986; Yamamoto et al., 2009) and simulated results of the liquefaction strength curve for Toyoura sand at relative density $D_r = 50\%$ and 90%. The simulated results correspond well with the test results.

6. Numerical model for analysis

6.1. FEM model

Fig. 5 shows the finite-element mesh used in this study. The soils were modeled with four-node isoparametric solid elements with 0.5 m x 0.5 m meshing. The metal plate at the top of the foundation was modeled in three equal horizontal layers. For all soil layers, the cyclic elastoplastic model was used for analysis. The elements above the water table were treated as dry elements without degrees of freedom of the pore fluid pressure. The metal plate was modeled by linear elastic elements. Soil in the desaturated zone was modeled by using

<table>
<thead>
<tr>
<th>Parameters</th>
<th>$D_r = 50%$</th>
<th>$D_r = 90%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density ($\rho$ (t/m$^3$))</td>
<td>1.87</td>
<td>1.96</td>
</tr>
<tr>
<td>Initial void ratio ($e_0$)</td>
<td>0.791</td>
<td>0.642</td>
</tr>
<tr>
<td>Coefficient of permeability ($k$ (m/s))</td>
<td>2.0E–5</td>
<td>1.5E–5</td>
</tr>
<tr>
<td>Compression index ($\lambda$)</td>
<td>0.0025</td>
<td>0.0091</td>
</tr>
<tr>
<td>Swelling index ($\kappa$)</td>
<td>0.000030</td>
<td>0.00052</td>
</tr>
<tr>
<td>Initial shear modulus ratio</td>
<td>1150</td>
<td>2023</td>
</tr>
<tr>
<td>Failure stress ratio ($M_f$)</td>
<td>0.99</td>
<td>0.99</td>
</tr>
<tr>
<td>Phase transformation stress ratio ($M_m$)</td>
<td>0.707</td>
<td>0.707</td>
</tr>
<tr>
<td>Hardening parameter ($B_0$)</td>
<td>3750</td>
<td>6000</td>
</tr>
<tr>
<td>Control parameter of anisotropy ($C_d$)</td>
<td>2000</td>
<td>2000</td>
</tr>
<tr>
<td>Reference strain parameter ($\gamma'$)</td>
<td>0.003</td>
<td>0.03</td>
</tr>
<tr>
<td>Reference strain parameter ($\gamma$)</td>
<td>0.005</td>
<td>0.36</td>
</tr>
<tr>
<td>Dilatancy parameter ($D_0$)</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Bulk modulus ($K_f$)</td>
<td>2.0E+5</td>
<td>2.0E+5</td>
</tr>
</tbody>
</table>
the same parameters as those of the saturated loose sand, with the exception of the bulk modulus. The bulk modulus for the desaturated zone was determined using Eq. (5). The determined bulk moduli for the desaturated zones of both lighter and heavier foundation models are summarized in Table 3. No slip was assumed in the horizontal directions between the structures and the soil.

6.2. Boundary conditions

Because a rigid container was used for the centrifuge model test, the bottom of the model was fixed and the lateral boundaries were fixed in the normal direction to the container wall. Regarding the drainage boundary condition, the lateral and bottom boundaries were impermeable, whereas the water table boundary on top was considered to be permeable. Recorded base acceleration in the centrifuge test (Fig. 6) was input at the rigid bottom boundary.

6.3. Numerical conditions

Time integration steps of 0.01 s were adopted to ensure numerical stability. Basic hysteresis damping by the constitutive model was used, and Rayleigh damping depending on the initial stiffness was used ($\alpha_0=0$ and $\alpha_1=0.001$, which corresponded to a damping ratio of less than 0.01 for the first predominant frequency of soil) in this simulation. The coefficients of the Newmark-$\beta$ method, $\beta$ and $\gamma$, were established at 0.3025 and 0.6, respectively, to ensure numerical stability.

7. Results and discussions

The centrifuge models were numerically simulated and results were compared with centrifuge tests in terms of the excess pore pressure ratio (EPPR), volumetric strain ($\varepsilon_v$) distributions, structural settlements, and deformation of the models following the shaking.

7.1. Excess pore pressure ratio

The time histories of the EPPRs, which are the ratios of excess pore pressure to the initial effective overburden pressure at each location, are given in Figs. 7 and 8. The increase in the observed EPPRs at all locations in the saturated centrifuge models (M1-1 and M2-1), nearly reaching unity in some cycles, indicates that the soil liquefied. The EPPRs obtained from numerical simulation were quite similar. For the desaturated models (M1-2 and M2-2), however, the EPPR responses were quite different from those for the saturated models. The observed EPPRs in the desaturated zone (B1, B2, C1 and D1) increased at a lower rate and remained significantly smaller than those in saturated models throughout the shaking events. Due to the differences in pore pressures between desaturated and saturated zones, pore fluid flowed and the boundary between the zones moved but these effects are not considered in the simulation. Some dissimilarity seen on the generation pattern of the excess pore pressures between the centrifuge test and the numerical simulation may be partly attributable to this limitation.

In the saturated zone of the desaturated models, the EPPRs at C2 and D2 remained significantly lower for most of the time during shaking than those in the saturated model. This clearly indicates that the pore pressure interactions between saturated and desaturated zones are due to pore fluid migration during the shaking. EPPRs at C2 and D2 were located close to the...
desaturated zone, and generated excess pore pressures were absorbed by the desaturated zone. The EPPRs of C3 located relatively far from the desaturated zone are similar to the saturated models. All these features observed in the desaturated centrifuge models were relatively well-replicated by the numerical simulation. Simulated pore pressures in the desaturated zone (B1, B2, C1 and D1) smoothly increased without fluctuation, which was also apparent in the saturated zone. The lower bulk modulus of pore fluid used for the desaturated zone in the simulation is responsible for this.

It is deduced from Eqs. (4) and (5) that the potential volumetric strain, the maximum volumetric strain when the excess pore pressure reached the initial vertical pressure, becomes smaller for the lower initial confining pressure. This suggests that the desaturation may not be very effective as a liquefaction countermeasure for lighter structures. However, although the soils tested in this study were in at relatively low effective stresses levels, desaturation was an effective way to mitigate excess pore pressure generation during earthquakes.

The distribution of maximum excess pore pressure ratios developed in the desaturated models is shown in Fig. 9. Despite the same degree of saturation, the maximum EPPRs in the desaturated zone (B1, B2, C1 and D1) were mitigated more effectively for the heavy load foundation model. A possible
the potential volumetric strain, which has a dominant effect on the liquefaction resistance of unsaturated soils (Okamura and Soga, 2006), was higher for M2-2 than M2-1. Quantitative discussions on the volumetric strain will be given in the next section. Other possible reasons are the effects of foundation weight on cyclic stress ratio and the anisotropic stress condition of soils below the foundations. These effects on the EPPRs can be seen not only for the response of desaturated models but also saturated models. An increase in the foundation weight enhanced both the initial vertical effective stress and the cyclic shear stress during shaking in the soil below the foundations, and consequently resulted in a decrease in the cyclic stress ratio. The existence of foundation also generates the anisotropic stress condition. Unlike a soil with level ground surface, the soil below the foundation has vertical stress being always higher than horizontal stress even during shaking and is not be able to reach the isotropic stress condition. The EPPR with the initial vertical effective stress as a reference stress decreases as an increase in the foundation weight. This stress anisotropy is the driving force of lateral spreading deformation of the soil and is more significant for the soil below heavier foundation.

7.2. Volumetric strain distributions

Fig. 10 shows the evolution of $\varepsilon_v$ at 4 m depth in the desaturated models (M1-2 and M2-2), obtained from numerical simulations. Volumetric strain is generally higher in the desaturated zone and close to zero in the saturated zone. At $t=12$ s, when the EPPR in the saturated zone (C3) of the models had already reached approximately unity and that in the desaturated zone (C1) remained smaller than 0.1, the volumetric strain remained very low. When $t=20$ s, EPPRs at C1 and C2 gradually increased to 0.2 and 0.6, respectively, and the volumetric strain at C1 and C2 increased accordingly whereas that in the saturated zone remained very low. Kazama (2006) and Unno et al. (2008) conducted undrained cyclic triaxial tests on unsaturated sand and found volumetric strains in direct proportion to excess pore water pressures. This is consistent with the numerical results obtained in the present study. According to Okamura and Soga (2006), a 1% volumetric strain almost doubles the liquefaction resistance of clean sands. Volumetric strain in the desaturated zone ranging between 0.6% and 1.5% may account for the significantly lower EPPR.
The difference in the volumetric strains in the desaturated zones of M1-2 and M2-2 became apparent as the shaking proceeded. The volumetric strains at $t=35\text{s}$ were larger for the model of the heavier load foundation (M2-2). This clearly indicates that the weight of foundation does have the effect on the volumetric strain, and thus the liquefaction resistance. Fig. 11 shows the volumetric strain distribution at the end of shaking ($t=35\text{s}$) for all four models obtained from numerical simulation. The saturated loose sand layers in Models M1-1 and M2-1 exhibited very low volumetric strains, except for the zones near the surface and bottom, where drainage of water occurred. On the other hand, for desaturated Models M1-2 and M2-2, volumetric strain in the desaturated zone was apparently higher than that in the saturated models, whereas the volumetric strain in the saturated zone was similar to that in the saturated models. The volumetric strain distribution patterns just below the steel plate differs from each other. This discrepancy may be due to the differences in the initial confining pressure imparted by the 10 kPa and 35 kPa steel plate placed at the top of the foundation.

Regarding the unsaturated surface layers, volumetric strains were generally higher than those in the saturated layers, and volumetric strains for the desaturated models were higher than those for the saturated models. As discussed in the next section, shear deformation of unsaturated surface layers was significantly larger for saturated models, and the soil in this layer showed dilation. This is believed to be responsible for the smaller contractive volumetric strain in the surface layers of saturated models.

7.3. Settlements and deformation

Settlement time histories obtained from the centrifuge tests and simulation are presented in Fig. 12. The simulated settlement agreed fairly well with that observed in the centrifuge tests. The settlements curve of the numerical model (FEM), M2-1 and M2-2 differed from the centrifuge model. Centrifuge test was conducted in 50$g$ centrifuge acceleration, small slide or tilt of structure placed on top of the foundation, can make a big difference (1 mm $= 5c \text{ma} t=50g$) on settlement measurements. This discrepancies may be due to slightly tilt of the structure during the shaking. Although foundation soils below structures in the desaturated model did not liquefy, settlement of several centimeters occurred. Considering that the sands in the 6-m-deep foundation ground volumetrically strained at 1%, this amount of settlement is inevitable.

![Fig. 11. Volumetric strain distribution ($\varepsilon_v$).](image1)

![Fig. 12. Foundation settlements at structure center.](image2)
Deformation of the models after shaking is demonstrated in Figs. 13 and 14. Overall deformation mechanisms of centrifuge models and those obtained from simulations were quite comparable in both the saturated and the desaturated models. In the saturated models, foundation soil directly below the structure was compressed vertically and spread laterally, and soil on the sides of the ground compressed laterally and the free ground surface heaved upward. However, for the desaturated models, deformation of the soil directly below the foundation was effectively suppressed by the desaturation. Soil in the saturated zone subsided owing to consolidation after shaking without lateral deformation. Ground surface settlement outside the foundation (saturated zone) was even larger than foundation settlement. This confirms that lowering the pore fluid bulk modulus of soil immediately below relatively light existing structures is sufficiently effective in mitigating structural settlement.
8. Conclusions

Highly instrumented dynamic centrifuge tests were conducted to study the performance of a new liquefaction countermeasure technique for the liquefiable foundation soil of relatively light structures. In this study, numerical simulations of the centrifuge tests were conducted to validate the numerical procedures and to further assure the effectiveness of this desaturation technique.

The numerical simulation attempted to simulate desaturated sand with an approximately 85% degree of saturation by reducing the bulk modulus of the pore fluid. The bulk modulus of the pore fluid, which is significantly lower than that of water because of air in the pores, was determined based on the degree of saturation and initial effective overburden pressures. Excess pore pressures as well as structural settlement and deformation mechanisms of foundation soils observed in the centrifuge tests were mostly accurately duplicated by the simulations.

Volumetric contractive strain of desaturated sand increased with increasing excess pore pressures during shaking. The structural settlement of the desaturated models was commensurate with the volumetric strain of soil below the structures.

This study confirmed that the seismic behavior of desaturated soil is successfully modeled by reducing the bulk modulus of pore fluid. Soil desaturation is sufficiently effective in mitigating the settlement of relatively light structures such as one- or two-story residential buildings.

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