EXPERIMENT ON GROUND SUBSIDENCE CAUSED BY INTERACTION BETWEEN UNDERGROUND STRUCTURE AND LIQUEFIED GROUND

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ABSTRACT

Local subsidence damage to an airport taxiway at Sendai Airport in Natori, Japan, occurred during the 2011 off the Pacific coast of Tohoku Earthquake. A shake table test of a large model in a 1 g gravitational field was performed to clarify the mechanism of this subsidence damage with a focus on the relationship between strain history and ground surface subsidence of the liquefied ground. The maximum and accumulated strain of the deviator shear strain close to the underground structure increased compared to other areas farther from the underground structure. As ground surface subsidence increased, the distribution of subsidence corresponded to the distribution of the maximum strain and accumulated strain in the ground. The results indicate that local subsidence damage at Sendai Airport was caused by interactions between the ground and the underground structure.

Keywords: airport pavement, earthquake damage, liquefaction, shake table test, underground structure

1. INTRODUCTION

At Sendai Airport, Natori, Japan, local subsidence damage to the airport taxiway over the ground near an underground structure occurred during the 2011 off the Pacific coast of Tohoku Earthquake (see Figure 1 and 2). Sendai Airport was the first local airport where liquefaction countermeasures were implemented in Japan. Installation of countermeasures began in 2008, and was scheduled to require about ten years for completion. However, the 2011 Tohoku earthquake occurred three years after construction began, and the airport was damaged by earthquake motion and the ensuing tsunami. Liquefaction of the backfill around the underground structure and original sandy ground at the taxiway was predicted, and liquefaction countermeasures were scheduled. However, the countermeasures were not finished before the earthquake. Subsidence damage occurred only around underground structures, which were box culverts for an underpass and waterway under the taxiway. The width of subsidence differed depending on the area, and the width of subsidence was narrower than the width of the backfill area in one area. The authors focused on the interaction between the ground and underground structure and hypothesized that local subsidence damage was caused by large shear strain of liquefied ground surrounding the underground structure.

In this study, a shake table test of a large model in a 1 g gravitational field was performed to clarify the mechanism of this subsidence damage, focusing on the relationship between strain history and ground surface subsidence of liquefied ground.

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Figure 1. Earthquake damage of taxiway at Sendai Airport in 2011 (Tsubokawa et al. 2012). (a) Local subsidence. (b) Welter

Figure 2. Ground improvement schematic of the taxiway at the time the earthquake occurred (Tsubokawa et al. 2012). (a) Box culvert for underpass. (b) Box culvert for waterway

2. SHAKE TABLE TEST CONDITIONS

2.1 Measurement setting and model creation

Figure 3 presents the model configuration and transducer arrangement. A rigid steel box (1.5 m height × 4.0 m width × 1.4 m depth) on an underwater shake table was used. The scale ratio (= prototype scale/model scale) of the length was set to 15 in accordance with the size of the box culverts at Sendai Airport. The height of the box culvert model and the thickness of the sand layer were 0.53 m. The similitude law in the 1 g gravitational field proposed by Iai (1988) was applied.

The base and sand layers were made from Iide silica sand No. 6 ($\rho_s = 2.641 \text{ g/cm}^3$, $\rho_{d_{\text{max}}} = 1.706 \text{ g/cm}^3$, $\rho_{d_{\text{min}}} = 1.417 \text{ g/cm}^3$). To prepare the base and sand layers, sand was respectively pluviated through air or poured into water stored in the pool after accelerometers and porewater pressure transducers were placed. Then, Iide silica sand No.4 ($\rho_s = 2.639 \text{ g/cm}^3$, $\rho_{d_{\text{max}}} = 1.782 \text{ g/cm}^3$, $\rho_{d_{\text{min}}} = 1.524 \text{ g/cm}^3$) was pluviated through air to prepare the surface layer. The relative densities calculated from the input weight of the base, sand, and surface layers were 44%, 44%, and 76%, respectively (see Figure 3). The box culvert model was fixed on the shake table in this test because displacement such as uplifting and sinking of the target was not measured.

Three vertical lines were set in the model, a point close to the box (Line 3), a point slightly to the east of the box (Line 2), and a point far from the box (Line 1) to evaluate the influential area of the interaction with the box (see Figure 3). Porewater pressure transducers and accelerometers were arranged along each line. Accelerometers were placed in the vertical and horizontal directions in each position to calculate the shear strain caused by the interaction between the ground and box in the sand layer (see Chapter 4).
2.2 Input acceleration waveforms

The input acceleration waveforms for the shake table tests were based on the level 2 earthquake motion defined at the engineering seismic base layer shown in Figure 4. A stepwise shake test (ratio of the amplitude of the original acceleration waveform increased 0.7 times, 1.0 time, and 1.5 times) was applied. The shake test steps are listed in Table 1 according to the maximum acceleration measured by the accelerometer installed on the shake table. The influence of sensor movement, deformation of the model, and increase in sand density were included after the second step, because the tests were performed in a stepwise manner. Therefore, the ground surface displacement and strain in the model mentioned in Chapters 3 and 4 are the incremental values of each step.

Table 1. List of shake test steps

<table>
<thead>
<tr>
<th>Magnification of amplitude</th>
<th>Max. acceleration (measured value)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 1</td>
<td>0.7</td>
</tr>
<tr>
<td>Step 2</td>
<td>1.0</td>
</tr>
<tr>
<td>Step 3</td>
<td>1.5</td>
</tr>
</tbody>
</table>

3. SHAKE TABLE TEST RESULTS

Soil liquefaction occurred immediately when shaking began in Step 1 under the smallest acceleration amplitude of the input waveform. No marked differences in the features of the excess porewater pressure and acceleration waveforms of the model were observed between Step 1 and Steps 2 and 3, with larger input waveform acceleration amplitudes. Therefore, only the time history results of Step 1 are presented.
3.1 Time history of acceleration

Figure 5 presents the time histories of acceleration corresponding to the locations in Figure 6. In the beginning, we focused on horizontal acceleration at Lines 1 and 2. The acceleration amplitudes in the base layer were amplified more than the input acceleration (H00). The acceleration amplitude in the sand layer became larger than the input acceleration until around 3 s after starting the shake test, and then suddenly decreased. This amplitude attenuation phenomenon in the sand layer is a feature of soil liquefaction and is caused by soil particles disengaging from soil liquefaction, resulting in shear force that cannot be transferred to the upper layer. Meanwhile, acceleration amplitude attenuation was confirmed at the upper part of the sand layer in Line 3 near the underground structure, but the degree of attenuation was smaller in the lower part of the sand layer, and there was a clear influence of the underground structure, which was fixed on the shake table and vibrated.

Figure 5. Time history of acceleration obtained at Step 1. (a) Horizontal direction. (b) Vertical direction
Next, we focused on the vertical acceleration records. Vertical acceleration measured on shake table was affected by the pitch of the shake table, and the maximum vertical acceleration was 134 Gal, approximately 40% of the horizontal acceleration (333 Gal). Because pitching caused by rotational movement of the shake table around its center, the vertical acceleration caused by pitching decreased closer to the center. The vertical acceleration in Line 1 amplified in the base layer and attenuated in the sand layer compared to the input acceleration, which was the same as the horizontal records. The vertical acceleration amplitude increased closer to the underground structure, and the vertical amplitude was larger than that of horizontal amplitude at Line 3.

Although the horizontal records indicated a tendency for attenuation due to soil liquefaction, the vertical records indicated a strong amplification tendency. This was because the sand layer moved vertically along the side of the underground structure, in a sloshing manner similar to the dynamic behavior of liquid storage tanks, because horizontal displacement of the sand layer differed from that of the underground structure fixed on the shake table. Consequently, vertical components predominated near the underground structure.

### 3.2 Time history of the excess porewater pressure ratio

Figure 7 presents the time histories of the excess porewater pressure ratio in the sand and base layers. The positions and labels correspond to those in Figure 6. The excess porewater pressure ratio is defined as the ratio of excess porewater pressure to the initial effective overburden pressure. Liquefaction is usually defined as the occurrence of an excess porewater pressure ratio of 0.95 or higher. Figure 7 also presents waveforms filtered with a low-pass filter of 4 Hz.
The excess porewater pressure ratio in the sand layer began to increase about 1.5 s after the shaking test started, and increased to approximately 1.0 at about 3 s, resulting in liquefaction. The timing of the occurrence of liquefaction corresponded to the attenuation of the horizontal acceleration amplitude. This supports the earlier discussion that the attenuation of the acceleration amplitude was caused by liquefaction in the sand layer. The shape of the excess porewater pressure time history and occurrence of liquefaction were similar at all position in the sand layer, although a substantial vibrational component was observed at Line 3 near the underground structure. The vibrational component may have been caused by hydrodynamic pressure, because this component was observed before liquefaction. Then, the influence of the cyclic mobility of sand may have been included in the vibrational component after the occurrence of liquefaction.

The occurrence of liquefaction in the base layer could be observed from the increase in the maximum value of excess porewater pressure ratio to approximately 1.0. However, the base layer, did not undergo complete liquefaction, because the excess porewater pressure ratio reached a maximum for only short durations and the acceleration time history in the base layer indicated an amplification tendency.

### 3.3 Subsidence of ground surface

Figure 8 shows the vertical displacement of the targets placed on the ground surface. The level of the target (measured in millimeters using a leveling staff) and vertical displacement were defined as the difference in the level before and after each step. The ground surface displacement distribution in Step 1 followed an arc shape with the underground structure at the center, because the surface layer immediately over the underground structure did not subside and displaced to the lateral direction following subsidence of the surrounding ground. Meanwhile, the displacement distribution after Step 2 contained discontinuous areas around the underground structure. These areas showed cracking caused by tensile failure of the surface layer at the boundary between the underground structure and the soil layer as marked differences in level (see Figure 8). Consequently, lateral displacement from the surface layer immediately over the underground structure decreased and settling increased locally near the underground structure. Overall, the local subsidence of the ground surface in the shake test was similar to the earthquake damage at Sendai Airport.

![Figure 8. Vertical displacements at targets placed on the ground surface (incremental value of each shake step)](image)

### 4. EVALUATION OF GROUND DEFORMATION

#### 4.1 Method for calculating strain history

The strain history of the ground was estimated from the acceleration time history obtained by the accelerometers to evaluate ground deformation. The simple shear strain $\gamma_\theta$ and deviator shear strain $\gamma_d$ corresponding to shear mode (see Figure 9) can be calculated with the following equations.
where $h$ is the distance between the accelerometers and $\ddot{u}(t)$ is the measured acceleration time history. The subscript number expresses the sensor number, and $x$ and $y$ express the horizontal and vertical directions of the acceleration components, respectively (see Figure 10).

Figure 9. Shear deformation mode in the ground (dashed line: before deformation). (a) Simple shear. (b) Deviator shear

The simple shear strain calculation was derived from the method used in Takahashi et al. (2011). Relative displacement at different depths was calculated by integrating the difference in acceleration between two accelerometers twice. Simple shear strain was obtained from the relative displacement divided by the distance between accelerometers. A band pass filter (lower limit: 0.5 Hz, upper limit: 50 Hz) was applied to the time histories of acceleration in advance. Deviator shear strain was calculated assuming undrained conditions (Poisson’s ratio = 0.5), because it was difficult to evaluate normal strain in the horizontal direction based on the arrangement of the accelerometers in this test. For example, when compression strains occurred in the vertical direction, we assumed that tensile strains occurred in horizontal direction, and deviator shear strain calculated as twice the value of vertical compression strain.

The time history of the maximum shear strain $\gamma_{\text{max}}$ was obtained using the following equation from the time histories of simple shear strain and deviator shear strain.

$$
\gamma_{\text{max}}(t) = \sqrt{\gamma_{xy}(t)^2 + \gamma_a(t)^2}
$$

Accumulated shear strain is defined as the accumulated absolute value of an increment of maximum shear strain at each time interval using:

$$
\gamma_{\text{cum}} = \int |d\gamma_{\text{max}}(t)|dt
$$

However, this equation cannot evaluate the change in shear strain generated by rotation of the
principle strain axis. Therefore, the increment in the maximum shear strain required in Equation 4 was obtained using:

\[
d\gamma_{\text{max}}(t) = \sqrt{\left(\gamma_{x}(t) - \gamma_{x}(t - dt)\right)^2 + \left(\gamma_{a}(t) - \gamma_{a}(t - dt)\right)^2}
\]  

(5)

Accumulated shear strain is an index correlated with the post-liquefaction volumetric strain of sand and is applied to evaluate settlement of the ground model in the shear box (Unno et al. 2012). Hereafter, maximum shear strain represents the maximum value of strains (i.e., the maximum shear strains (Equation 3) and accumulated shear strain (Equation 4)) in the time history.

4.2 Relationship between the distribution of strain in the ground and ground surface subsidence

Figure 11 presents the distributions of both the accumulated and maximum shear strains. Figure 12 presents the distribution of vertical displacement measured by the laser displacement transducers. The three plots located in the horizontal direction correspond to the results at Lines 1, 2, and 3 (see Figure 6). The shear strain results were the mean values of the three areas between four accelerometers in the sand layer in the vertical direction. The mean values were averaged and weighted based on the thickness of the layer between each accelerometer.

Figure 11. Distribution of the accumulated and maximum values of shear strain

Figure 12. Vertical displacement at the ground surface measured by laser displacement transducers
The distributions of both the accumulated and maximum shear strains followed the same trends. Both strains close to the underground structure (Line 3) became more than two times larger than those in the area far from the underground structure (Line 1). At Line 2, which was slightly east of the structure, the influence of the interaction with the underground structure was small and the strains were similar to those of Line 1. The distributions of vertical displacement in Steps 2 and 3, but not Step 1, where the influence of displacement occurred in the lateral direction of the surface layer, were consistent with distributions of the estimated shear strains in the ground.

Figure 13 presents the distribution of the accumulated and maximum values of the simple shear strain component and deviator shear strain component, respectively. In all steps, the simple shear strain component decreased slightly near the underground structure and the deviator shear strain component increased markedly near the underground structure. Therefore, local subsidence caused by volumetric strain of sand became large due to the accumulated shear strain and dissipation of excess porewater pressure near the underground structure.

Figure 13. Comparison of simple shear strain and deviator shear strain. (a) Distribution of the maximum value. (b) Distribution of the accumulated value
The area affected by soil liquefaction and the amount of excess porewater pressure are usually estimated to predict airport pavement settling in practical designs; however, the results indicated that this prediction method cannot evaluate local settlement near underground structures. Evaluating the strain history of liquefied ground considering seismic interactions between the ground and underground structures is important for the accurate prediction of local ground surface subsidence.

5. CONCLUSIONS

In this study, a shake table test was performed to clarify the relationship between the shear strain history and ground surface subsidence of liquefied ground. The results showed that both the accumulated and maximum values of deviator shear strain in the sand layer near the underground structure increased in comparison to other positions farther from the underground structure. As ground surface subsidence increased, the distribution of subsidence corresponded to the distribution of the shear strain history in the ground. The results indicate that the local subsidence damage at Sendai Airport may be have been caused by interactions between the ground and the underground structure. The authors are also attempting to perform laboratory testing for evaluating volumetric strain properties of liquefied sandy soil due to accumulated shear strain. The experimental results will be validated by the results of the numerical analysis considering these properties.

6. REFERENCES


