Physical Modelling of Earthquake-induced Liquefaction on Uniform Soil Deposit and Settlement of Earth Structures

Avantio Pramaditya¹,², Teuku Faisal Fathani¹,²,*
¹Department of Civil and Environmental Engineering, Universitas Gadjah Mada, Yogyakarta, INDONESIA
Jalan Grafska No 2 Yogyakarta
²Centre of Excellence for Disaster Mitigation and Technological Innovation GAMA-InaTEK, Universitas Gadjah Mada, Yogyakarta, INDONESIA
Jalan Grafska No 2 Yogyakarta
*Corresponding authors: tfathani@ugm.ac.id

ABSTRACT Earthquake-induced liquefaction has been known as a complex and challenging topic in the field of geotechnical engineering. The phenomenon could bring catastrophic damage as has been seen from the past with severe damage seen on the ground and various structures such as buildings, earth structures, and important lifelines structures. The occurrence of liquefaction is caused by the loss of strength and stiffness of the cohesionless saturated soils due to the rapid dynamic loads from the earthquake. In order to analyse and observed the earthquake-induced liquefaction phenomena, physical modelling subjected to geotechnical centrifuge test was conducted in this study. The study aims to understand the liquefaction phenomena, mechanism and consequences through physical modelling by centrifuge test and laboratory testing. Embankment lies on liquefiable foundation ground was modelled by means of physical modelling and subjected to the earthquake motion of The 2011 Tohoku Earthquake retrieved from K-Net Mito stations. Geotechnical centrifuge test with 50 g of centrifugal acceleration was conducted in order to create the conditions of the actual field. The behaviour of the model was observed using sensors for acceleration, pore pressure, and displacement. Liquefaction manifestation could be seen in the model with the occurrence of lateral spreading, remnants of the sand boils, and deformation of the embankment. The rapid development of excess pore water pressure occurred, and pore pressure ratio (rₚ) higher than 1 indicated the occurrence of liquefaction. The observed settlement of the embankment is around 0.43 m.

KEYWORDS Excess Pore Water Pressure; Geotechnical Centrifuge Test; Liquefaction; Physical Modelling; Settlement.

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1 INTRODUCTION

The earthquake has been known as one of the most disastrous phenomena that could bring devastating damage to the surrounding area. An aftermath event could follow the earthquake, that could bring more damage, such as liquefaction. Liquefaction is one of the most important and interesting topics in geotechnical engineering (Kramer, 1996). The occurrence of the Niigata earthquake and the Alaska earthquake in 1964 that demonstrate the great scale of the liquefaction effects and damages emphasise the importance of liquefaction. Hundreds of buildings are sinking and tilting with more than 50% of damaged shallow and piled foundation classifies as intermediate and heavy damage during the 1964 Niigata earthquake (Kishida, 1966). Spectacular mud spouts, subsidence and numerous ground cracks were observed during preliminary evaluation after the Alaska earthquake (Grantz, Plafker and Kachadoorian, 1964). Those earthquake-induced liquefaction events also trigger the geotechnical engineers in conducting studies and research specifically in liquefaction. The more recent notable earthquake-induced liquefaction is the Great Hanshin Earthquake in 1995 (Kitagawa and Hiraishi, 2004), the Canterbury Earthquake Sequence in 2010-2011 (Cubrinovski et al., 2010); (Yamada et al., 2011), the 2011 Tohoku Earthquake (Towhata et al., 2014), and Palu earthquake in 2018 (Kiyota et al., 2020).

Liquefaction occurs in saturated cohesionless loose sandy soils that unable to maintain it strength and stiffness because of dynamic loading caused by the earthquake. During this process, rapid loading of
earthquake motion triggers the development of pore water pressure, decrease the effective stress (Towhata, 2008). The relation between effective stress and pore water pressure is as follows:

**Effective stress = Total Stress (weight of overburden soil) – pore water pressure**

The increase in excess pore water pressure (EPWP) generated due to the ground shaking could cause an upward flow that might result in the liquefied condition of the ground. This condition happens when the effective stress becomes zero, and the contact of the soil particles are released as if the soil particles are floating in the water (Ishihara, 1985).

In general, the damage caused by liquefaction are combinations of one or more ground failure. The ground failure consists of several forms which are sand boils, flow failures, lateral spreading, and settlements (Towhata, 2008). The possible manifestation of liquefaction depends on several aspects such as site conditions, the characteristics of the earthquake, and the structural properties. Idriss and Boulanger (2008), emphasise three types of liquefaction manifestation which often encountered and essential in practice. Those three types are, loss of strength; lateral spreading; and settlement. The importance of those three types of damages could be seen from the case histories. As shown in a study conducted by Cubrinovski et al. (2012), the typical damaged encountered in the residential foundation was differential settlement resulting in the permanent tilt of the residential house. Earth structures such as embankment, levee, and river dike were also subjected to liquefaction-induced damages. Several case histories of liquefaction-induced damages to the earth structures have been studied. Takada et al. (1996) study the liquefaction-induced damages to the river levees and embankment during the Kobe Earthquake. The study found that damage was concentrated at the soft alluvial sandy subsoils where a lot of river levees are located. Green et al. (2011) analysed the performance of levees during the Darfield Earthquake in 2010-2011 in which varying damages was found at the levee along the Waimakariri River and the Kaiapoi River due to the liquefaction in the foundation ground.

Due to the complexity and difficulties in analysing and assess the liquefaction phenomenon, numerous researchers have conducted studies and research regarding liquefaction through laboratory testing using physical modelling and geotechnical centrifuge test (Sharp, Dobry and Abdoun, 2003); (Elgamal et al., 1996); (Adalier, Elgamal and Martin, 1998). By using the centrifuge test, it is possible to recreate the actual field conditions of soil stress and strains in a scaled physical model (Ng, 2014). Numerous efforts have been made to improve the quality of the geotechnical centrifuge testing. Hushmand et al., (1988) conduct a centrifuge test of liquefaction using a newly constructed laminar box that accommodates the reduce in friction between adjacent layers of the box and increase the degree of accuracy of the data measurements. The Liquefaction Experiment and Analysis Project (LEAP) is an international collaborator project with the objective to provide a high-quality database of centrifuge test that can be used in assessing and validating constitutive models and techniques for analysis and mitigation (Manzari et al., 2018). The results of the centrifuge tests conducted as a part of LEAP projects are presented by Kutter et al. (2020).

In this study, physical modelling by centrifuge test was conducted in order to model the behaviour of earthquake-induced liquefaction to the earth structure on a laboratory scale. The centrifuge test was conducted to achieve the stress and strain of the actual field conditions. Thus, the results from the centrifuge test should be a representation of the actual field conditions. The model subjected to centrifuge test is observed and analysed throughout this study in order to enhance the understanding of the liquefaction mechanism and its consequences.
2 METHODOLOGY

2.1 Model Preparation

Embankment lies on liquefiable foundation ground was modelled by physical modelling. In conducting a physical modelling and centrifuge test, it is important to have controlled materials and conditions. Kutter et al. (2020) presented the model specification of the LEAP projects, which shows that the setup and modelling process should be conducted with caution in order to achieve reasonable test results. Fabricated soils were used as materials for foundation ground and embankment model. The benefit in using fabricated soils is the ability to have a well-controlled model as they have controlled soils properties. The foundation ground was constructed using Toyoura sand, well-known Japanese test sand, which has been studied by numerous researchers (Koseki, Yoshida and Sato, 2005; Tatsuoka et al., 1986; Cubrinovski, 2011). Previous studies have outlined that mechanical properties of Toyoura sand in dry and saturated state is similar. The embankment was made with DL Clay mixed with silicon oil. DL is the abbreviation from driftless, and DL Clay is made from kaolin and silica stone with an average particle size of 28. Index properties of Toyoura sand and DL Clay are shown in Table 1.

| Toyoura Sand | DL Clay |
|--------------|---------|
| Density, \( \rho_s \) (g/cm\(^3\)) | 2.65 | Specific gravity (G.) | 2.65 |
| Mean particle size, \( D_{50} \) (mm) | 0.19 | Liquid Limit, LL (%) | NP |
| Particle size, \( D_{10} \) (mm) | 0.14 | Plastic Limit, PL (%) | NP |
| Maximum void ratio, \( e_{max} \) | 0.973 | Plasticity Index, IP | NP |
| Minimum void ratio, \( e_{min} \) | 0.609 | Coefficient of Permeability (m/s) | 9.0×10\(^{-8}\) |

Figure 1 shows the configuration of the model. The model was built on a rigid rectangular box made from steel which has a transparent side used to observe the model. The container has a length of 600 mm, a width of 240 mm, and a height of 400 mm. The height of the foundation ground is 220 mm. The groundwater table was set at 10 mm below the surface ground. The crest of the embankment is 80 mm and has 40 mm height with a slope of 1:1.5. Three types of instrumentation were installed at the foundation ground. Accelerometers placed in the model with the increment of 30 mm, pore pressure transducer (PPT) installed between the accelerometers. The free field consists of PPT 3; PPT 7; and PPT 11. The PPT 4; PPT 8; and PPT 12 placed beneath the tip of the crest area. The centre of the embankment was monitored by PPT 5; PPT 9; and PPT 13. PPT 6; PPT 10; and PPT 14 were beneath the toe of the embankment. Linear variable displacement transducer attached on the tip of the crest and the centre of the embankment. All of the dimension mentioned above are on the model scale. Details of the model are shown in Table 2.

| Materials | Foundation ground | Embankment |
|-----------|------------------|------------|
|           | Toyoura sand, relative density \( (D_r) \) of 50% | DL Clay mixed with silicon oil; slope of 1:1.5 |
| Groundwater table | 0.5 m | |
| Accelerometers | 9 points | 3 points |
| Pore pressure | 13 points | - |
| Displacement transducer | - | 3 points |
The liquefiable foundation ground was prepared to have a relative density ($Dr$) of 50% based on the previous studies conducted by Maharjan and Takahashi (2014). The preparation procedure was conducted in a similar way with previous centrifuge test that has been conducted (Adamidis and Madabhushi, 2018; Kutter et al., 2020; Maharjan and Takahashi, 2014). In order to achieve the desired relative density, air pluviation method was used (Tabaroei et al., 2017). The sand was poured using a hopper from above the container, back and forth, evenly. The desired density is achieved by keeping the falling height of the sand constant. Calibration of the falling height has been performed before the model preparation. The embankment was made manually by shaping the mixture of DL Clay with silicon oil with a ratio of 1:2. The usage of silicon oil on the embankment mixture is for the sake of easier shaping of the embankment. The shaping of the embankment was done directly on the container after the foundation ground preparation has been done.

2.2 Centrifuge Test

Centrifuge test has the capability to recreate the actual soil conditions into the scaled laboratory test model. It enables the possibility to tackle and solve complex geotechnical problems through a laboratory test. Thus, this method generally used to assess and validate the mechanism and behaviour of the soil (Youd, 1995), and also for validation of constitutive models (Manzari et al., 2015). In order to replicate the actual stress and strains, the scaled model, $1/N$, is subjected to centrifugal acceleration, $N\cdot g$, to accommodate the differences in confining pressures (Gopal Madabhushi, 2007). The scaling law for geotechnical centrifuge test presented by Schofield (1981) is shown in Table 3.

The centrifuge test in this test was conducted using the Tokyo Tech Mark III centrifuge machine from the Department of Civil Engineering, Tokyo Institute of Technology. The schematic figure of the centrifuge machine is shown in Figure 2. The model was placed at the swinging basket and counterweight was placed at the other side to stabilise both of the centrifuge arms. There are two electrical slip rings, one for recording the instrumentation data and the other one is for operation. 50 g of the centrifugal acceleration was applied to the model.
Table 3 Scaling law for geotechnical centrifuge test (after Schofield, 1981)

| Parameter                    | Scaling law |
|------------------------------|-------------|
| General                      |             |
| Length (m)                   | 1/N         |
| Area (m²)                    | 1/N²        |
| Mass                         | 1/N³        |
| Volume (m³)                  | 1/N³        |
| Stress                       | 1           |
| Strain                       | 1           |
| Time (consolidation)         | 1/N²        |
| Dynamic condition            |             |
| Velocity                     | 1           |
| Acceleration                 | N           |
| Frequency                    | N           |
| Displacement                 | 1/N         |
| Time (dynamic)               | 1/N         |

The earthquake load of the 2011 Tohoku earthquake was used in the centrifuge test. Liquefaction consequences were found in levees along Naka river in Mito City (Towhata et al., 2013) thus, the NS component of input motion was retrieved and used from K-Net stations located at Mito. K-Net or Kyoshin-Network, managed by National Research Institute for Earth Science and Disaster Resilience (NIED), is a comprehensive strong-motion seismograph network with approximately 20 km of an interval between the stations (Suzuki et al., 2017). It is free to access and could be accessed by the public. Figure 3 shows the input motion used in this study. Due to the motion recorded at the surface, it is assumed that the magnitude of the bedrock motion is 70% of the surface motion. The maximum acceleration of the input motion in this study is 0.29 g.
Figure 3 Input motion of the 2011 Tohoku earthquake used in this study.

3 RESULTS

3.1 Visual Assessment

Figure 4 (a) shows the side view, and Figure 4 (b) shows the top view of the model conditions after subjected to the centrifuge test. The black dash line is the initial shape of the embankment, and the red dash line is the initial contour line. Embankment underwent deformation, as shown in the figure. The settlement of the embankment occurred partially as a result of lateral spreading occurred at the foundation ground. The lateral spreading of the foundation ground is indicated by the bends of the contour line beneath the embankment towards the free-field area (yellow marker) Figure 4 (a). The foundation ground beneath the embankment sinks slightly as can be seen from the upheaving of the foundation ground near the toe of the embankment (yellow circle). In Figure 4 (b), cracks are found at the top of the embankment. The remnants of sand boils could be seen at the foundation surface near the embankment. The fine-grained at the surface might contribute to the sand boils occurrence, as presented in a study conducted by Scott and Zuckerman (1972).

Figure 4 Model condition after subjected to the centrifuge test; (a) Side view; (b) Top view

3.2 Excess Pore Water Pressure

Excess pore water pressure was obtained from the installed PPT at the foundation ground. As could be seen from Figure 1, the PPT was installed in 4 different areas at the foundation ground which is free field, beneath the tip of the embankment crest, beneath the centre of the embankment, and beneath the toe of the embankment. The configurations of EPWP make the observation of the embankment effects towards EPWP generation compared to the free field possible. The EPWP generated from the model is shown in Figure 5.
Figure 5 (a) shows the EPWP generation at 1.5 m depth of the model. The EPWP starts to develop at around 187 s for all of the location. It rapidly increases to the value of 18 kPa and going steady after reaching its maximum value. However, EPWP from beneath the crest and the centre of the embankment slightly dissipate before it is increased to the similar value of the other location. Based on Figure 6 (a), liquefaction did not occur as the generated EPWP is lower than the pore pressure ratio, $r_u=1$.

Figure 5(b) shows the EPWP generated at 3 m depth. The EPWP developed until around 30 kPa and going steady except for the location beneath the toe of the embankment. The EPWP at that location developed until around 65 kPa. This condition was not found at the 1.5 m depth. It also could be seen that the location beneath the crest tip and the centre of the embankment have similar EPWP development as the location at the 1.5 m depth. Based on Figure 6 (b), liquefaction occurs as the generated EPWP is near the pore pressure ratio, $r_u=1$.

Figure 5 (c) shows the development of EPWP at 6 m depth. The transducers located beneath the tip of the crest have not recorded any value of EPWP development. The EPWP generated at other location has a similar value which developed until around 55 kPa. Based on Figure 6 (c), liquefaction occurs as the generated EPWP exceeds the pore pressure ratio, $r_u=1$.

### 3.3 Settlement

Settlement of the embankment was obtained from the LVDT placed at the centre and the tip of the crest. Based on Figure 6, a significant amount of settlement occurred in a short period when the earthquake loading increase rapidly. The embankment starts to deform at around 180 s and reach maximum value before going into steady-state condition. It shows that the settlement after the shaking was minimal. There is no significant difference between the settlement found on the tip and the centre of the crest. This finding is in line with Rapti et al. (2018) which explain that the depth of the liquefiable layer holds vital roles in determining the seismic response of the embankment. When the liquefiable layer located at the deep layer, there is no significant response found on the embankment.
Figure 5 Excess pore water pressure generated at (a) 1.5 m depth, (b) 3 m depth, and (c) 6 m depth.

Figure 6 settlement at the tip and the centre of the crest of the embankment

4 DISCUSSION

Liquefaction consequences on the foundation ground and earth structures could be seen from the centrifuge test conducted on the physical model of embankment placed on the liquefiable ground. Based on the visual assessment, several liquefaction manifestations are visible both on the foundation ground and embankment. There are a sign of lateral spreading occurrence at the foundation ground, deformation of the foundation ground, and settlement of the embankment. This phenomenon is in line with has been found from the case histories. Damage patterns of river embankments compiled by Oka et al. (2012) shows that typical damage patterns of earth embankment are lateral expansion, settlement, sand boils and cracks. Sasaki et al. (2012) found that the levee along Tone River in Japan undergone 1 m
of subsidence and crack was found at the crest of the levee and significant distortion was found at the Hinuma levee. Similar damage patterns also found in the 1988 Armenia earthquake (Yegian et al., 1994).

A sudden increase in EPWP was found in every location at the foundation ground (Figure 7). The highlighted area shows that EPWP starts to increase during the rapid dynamic loads of the earthquake. The maximum value of the EPWP was achieved during the highest peak ground acceleration. In general, free field area and foundation ground beneath the embankment generate a similar response in terms of EPWP. However, the EPWP at 5m depth beneath the toe of the embankment developed until around 65 kPa, much higher compared to the other location at the same depth. It also has different behaviour while compared to the same location at a different depth. The reason for this might be an error in data recording at the location mentioned above. The ideal theory of liquefaction occurrence is when pore pressure ratio, $r_u$, is 1. However, based on the strain-controlled undrained cyclic simple shear test conducted by Hazirbaba and Rathje (2004), when $r_u$ reaches a value greater than 0.9, it could be assumed that liquefaction happens. Based on the test results, liquefaction occurred at 6 m depth and 5 m depth at the free field, and at 6 m depth at the foundation ground beneath the embankment. Liquefaction did not occur at the shallow part of the foundation ground. The results obtained in this study is in line with the study presented by Adalier et al. (1998) and Maharjan and Takahashi (2014), which observed a stiffer response beneath the embankment. However, the shallower part of the free field area undergoes liquefaction.

In terms of the dissipation rate of the EPWP, every location has different dissipation rate and process. The EPWP at 6 m depth dissipates quickly starting from around 255 s. At 3 m depth, the EPWP starts to dissipate at around 310 s while the depth of 1.5 m took the longest to dissipate starts at around 340 s. The dissipation rate and time are governed by two main factors which are the duration of the seismic and drainage conditions (Day, 2002). Considering the same input motion were applied to the model, the drainage conditions of the foundation ground might be the reason for the differences in dissipation time. The shallowest depth at 1.5 m was confined by the embankment which constructed from clay materials. This condition makes the dissipation process longer compared to the deeper depth, which has liquefiable soil on the upper and lower layer. Maharjan and Takahashi (2014) conducted centrifuge testing for models with uniform and non-uniform soil deposit. The results show that the uniform deposits dissipate faster than the non-uniform deposits.

The settlement observed at the tip of the crest and the centre of the embankment is 0.43 m. and 0.42 m, respectively. The settlement at the centre is slightly smaller than at the tip of the crest indicating that the deformation in shape was minimum and the settlement of the embankment was a consequence of the liquefied foundation ground. Adalier et al. (1998) and Elgamal et al. (2002) observed similar conditions in which settlement occurred at a nearly uniform rate and the settlement of the embankment was due to the lateral spreading of the foundation ground. Additional settlement observed after the shaking was minimal.
Figure 7 Time histories of excess pore water pressure from (a) 1.5 m depth, (b) 3 m depth, and (c) 6 m depth, and (d) input motion.

5 CONCLUSION

A model of embankment placed on the liquefiable uniform ground deposits was subjected to the centrifuge test on 50 g of gravitational acceleration. The test aims to replicate the actual stress and strains conditions on a laboratory scale through physical modelling by a centrifuge test. The behaviour of the model was observed and analysed in this study. The visual assessment of the model conditions after the centrifuge test shows the signs of liquefaction occurrence on the foundation ground. The liquefaction manifestation that was observed from the model are lateral spreading, the stretch of the embankment, settlement, and cracks at the top of the embankment. The remnants of sand boils could also be seen on the ground surface. Liquefaction occurred at the free field at 6 m and 3 m depth, and at 6 m depth for the foundation ground beneath the embankment indicated by the development of excess
pore water pressure that exceeds the pore pressure ratio, $r_u = 1$. A similar amount of settlement was recorded at the centre and the tip of the crest. The recorded settlement is 0.45 m at the tip of the crest and 0.42 at the centre of the embankment. Based on the analysis, the cause of the settlement was partially due to the lateral spreading of the foundation ground and the settlement observed after the shaking was minimal.

**DISCLAIMER**
The authors declare no conflict of interest.

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