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GEOTECHNICAL CENTRIFUGE TEST OF REINFORCED ROAD EMBANKMENT AGAINST EARTHQUAKE-INDUCED LIQUEFACTION

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ABSTRACT: Liquefaction is one of the causes of ground and structural damages during the earthquake. The occurrence of liquefaction during the earthquake is caused by the contractive behavior of loose sands subjected to cyclic loading. Due to this phenomenon, the soil deposits will lose its strength, causing deformation and settlement. In this study, the seismic response of the road embankment with proposed countermeasures is analyzed by conducting a centrifuge model test. The model was subjected to a gravitational acceleration of 50g to replicate the prototype scale's actual conditions of stress and strain. Two types of proposed countermeasures, gravel mat and gravel mat with geogrid, were used in this study. The study aims to observe the behavior and mechanism of the model subjected to liquefaction. The results show that the proposed countermeasures helped prevent structural failure by reducing the excess pore water pressure development and its dissipation time and reducing the settlement. It also reduces the lateral spreading of the foundation ground based on visual observations.

Keywords: Embankment, Geogrid, Gravel mat, Liquefaction countermeasures, Physical modeling

1. INTRODUCTION

The earthquake has been known as a natural disaster that could cause catastrophic damage to the ground and structures above it. Earthquake is also known for the liquefaction phenomenon, the aftermath event that can be as destructive as the earthquake itself. Liquefaction began to catch attention in 1964 with the Alaska earthquake and Niigata Earthquake occurrence in within three months. Both earthquakes show the significance of the earthquake-induced liquefaction damages to the surrounding area [1, 2]. Liquefaction was also the cause for major destruction in recent earthquakes, such as the Canterbury Earthquake Sequence on 2010-2011 [3], the 2011 Tohoku earthquake [4,5]. In Indonesia, several notable earthquake such as 2006 Yogyakarta earthquake [6], 2009 Padang Earthquake [7], and Palu Earthquake and Tsunami in 2011 [8] reported to have manifested damages due to liquefaction.

Liquefaction has various consequences that might affect the structures and the surrounding area depending on the site condition, the earthquake loading characteristics, and the nature of the structures or the surrounding area [9]. One of the structures that could be affected by liquefaction is earth structures such as embankments, levees, or river dikes. Public Works Research Institute (PWRI) Japan, in their manual, classifies embankment failure due to earthquake into four types of damage modes; failure at the slopes, failure of the embankment, failure of the embankment and foundation ground, and subsidence of the embankment [10].

Several prominent studies have reviewed liquefaction cases in earth structures. For instance, during the Darfield and Christchurch earthquake in 2010 and 2011, Green et al. [11] observed the performance of the levee (stopbanks) and found several sections of levee that have cracks greater than 1 m deep, shows indication of deep-seated movement and/or settlement, lateral spread and 500 mm or more deformation of the levee. Oka et al. [12] conduct an in situ research after the 2011 Tohoku Earthquake to define the typical embankment damage patterns due to liquefaction. Heaving and settlement of the embankment, lateral expansion of the toe, longitudinal cracks, lateral movement of the slope, and fissure of the embankment body due to cracks.

In order to reduce and minimise the effect of liquefaction on the earth structures, various mitigation methods for liquefaction countermeasures has been developed. In general, mitigation methods aim to minimise the development of excess pore water pressure. Japanese Geotechnical Society in 1998 [13], Yasuda and Harada [14] classified the methods into two categories: soil improvement and structural strengthening. Several methods of mitigation methods are Impact methods such as vibro compaction and vibro replacement, gravel drains, jet grouting, and compaction piles [15].

In his study, Sasaki et al. [16] explained the application of geogrid sheets as remedial measures for liquefaction on the Arashima dike section. The reinforced embankment has less damages with no deformation observed and 20 cm of settlements observed after the Tottori-ken, Seibu Earthquake occurrence.

Centrifuge modelling is a major instrument in geotechnical engineering that enables the study and analysis of geotechnical problems by using geotechnical materials. The development of centrifuge modelling in geotechnical engineering has grown rapidly between 1980 to 1990 [17]. Widespread usage of centrifuge modelling in geotechnical application was apparent during those years. The advantage of centrifuge modelling is the availability to replicate the prototype stress and strains in the scaled physical model. It enables to recreate the realistic behaviour of geotechnical problems in the laboratory test. Physical modelling using centrifuge test is usually used for understanding the mechanism and behaviour of soil as well as validation. It is rarely used for designing purposes [18]. Several studies have been conducted using centrifuge modelling in regards to understanding mechanism and validation [19-21].

In 1981, Schofield [22] presented the scaling principle for dynamic earthquake geotechnical centrifuge models. Generally, scaling laws can be derived from dimensional analysis, differential equations, or the mechanical similarity between a prototype and a model. The scaled model, 1/N, is subjected to centrifugal acceleration, N·g, which makes the stress and strain in both media to be the same. Using instruments such as transducer or accelerometer, observation of the behaviour of the model before, during, and after the failure could be done [23]. However, the scaling laws and scaling errors are the two key issues in centrifuge modelling as it is an important factor in creating similar conditions between the prototype and the model. Most recent study has tried to validate the generalised scaling law in centrifuge testing [24].

This paper discussed a model of mitigated embankment resting on homogeneous liquefiable soil. There are two proposed methods of mitigation that are installed as a countermeasure in this study; gravel mat, and gravel mat with geogrid. The research significance of this study is to observe and analyse the mechanism of liquefaction through scaled physical modelling and centrifuge test. The behaviour of the model subjected to dynamic loading and the effect of the proposed countermeasures in reducing the damages of the embankment due to earthquake-induced liquefaction were also observed and analysed during this study.

2. EXPERIMENTAL FRAMEWORK

The model was constructed in a rectangular rigid container box which has a transparent front side with a dimension of 60×25×40 cm of length, width, and height, respectively. The model has identical conditions except for the mitigation method that is installed beneath the embankment. The other identical aspects are the foundation grounds, groundwater table, input motion, and materials.

In dynamic laboratory testing, it is important to understand the cyclic behavior and liquefaction resistance of the material [25]. Toyoura sand, the material used in this study, is a standardized sand in Japan that has been widely used as material for the purposes of laboratory testing in the geotechnical engineering field. The Toyoura sand has shown a tendency as a liquefiable material and has been used in laboratory testing and centrifuge experiments related to earthquake-induced liquefaction [19,21]. The gravel mat used as mitigation are prepared using the silica No. 3. Index properties of Toyoura sand and silica No. 3 are shown in table 1.

Table 1 Index properties of Sand materials

Item	Toyoura Sand	Silica No. 3
Density, ρ_s (g/cm^3)	2.65	2.56
Mean particle size, D_{50} (mm)	0.19	1.47
Particle size, D_{10} (mm)	0.14	1.21
Maximum void ratio, e_{min}	0.973	0.971
Minimum void ratio, e_{max}	0.609	0.702
Relative density, Dr (%)	50	-
Coefficient of uniformity, U_c	-	1.26

In order to capture and observe the response of the model, three types of instrumentations; Piezoelectric accelerometers, Pore pressure transducer (PPT), and Linear variable displacement transducer (LDVT), were installed in the model. The configuration of the model is shown in Figure 1. The PPT 3, PPT 7, and PPT 11 are located in the free field. The behavior of the foundation ground beneath the center of the embankment is monitored by the PPT 5, PPT 9 and PPT 13, and for the foundation ground beneath the toe of the embankment is observed by PPT 6, PPT 10, and PPT 14.

The foundation ground made from Toyoura sand was prepared using air pluviation method [26, 27] to obtain the desired relative density of approximately 50%. In order to comply with the scaling law of the centrifuge test, increasing the viscosity of the pore fluid was needed [28]. A mixture of deionized water and 2% of hydroxypropyl methylcellulose by weight of water is used to achieve viscosity that fulfilled the scaling law. The saturation of the model was conducted by vacuum saturation method.

The scaling law for geotechnical centrifuge modelling for this study is shown in Table 2. The scaling law used in this study was based on the scaling principle proposed by Schofield in 1981 [22]. The entire centrifuge test conducted at a centrifugal acceleration of 50g with input motion of the 2011 Tohoku earthquake retrieved from K-Net Mito station. It is assumed that the model consolidated during the preparation process The comparison of the input motion applied in each of the tests is shown in Figure 2. Test-1 is for the model with gravel mat and Test-2 is for the model with gravel mat with geogrid as proposed mitigation. The Tokyo Tech Mark III centrifuge machine was used in this study, shown in Figure 3.

Table 2 Scaling law for geotechnical centrifuge modelling used in this study

Parameter	Scaling law	Prototype	Scaled model (N=50g)
Length/displacement	1/N	1	1/50
Stress	1	1	1
Strain	1	1	1
Velocity	1	1	1
Acceleration	N	1	50
Frequency	N	1	50
Time (dynamic)	N	1	50

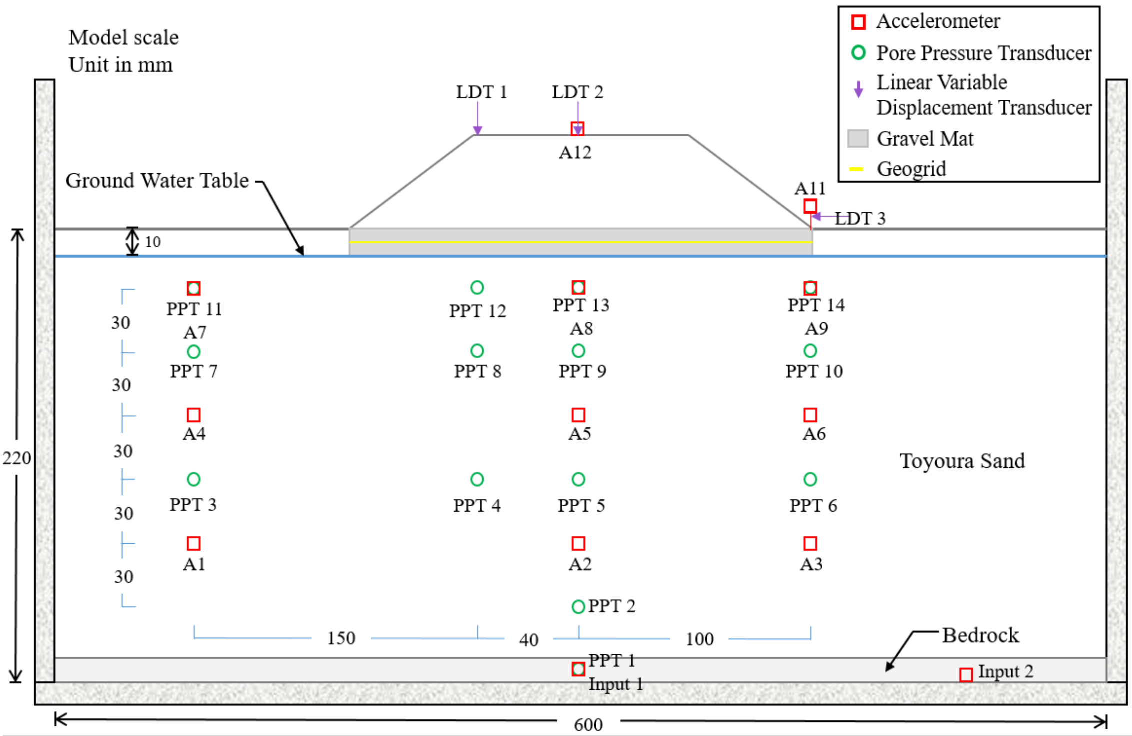

Fig. 1 Model configuration for centrifuge test

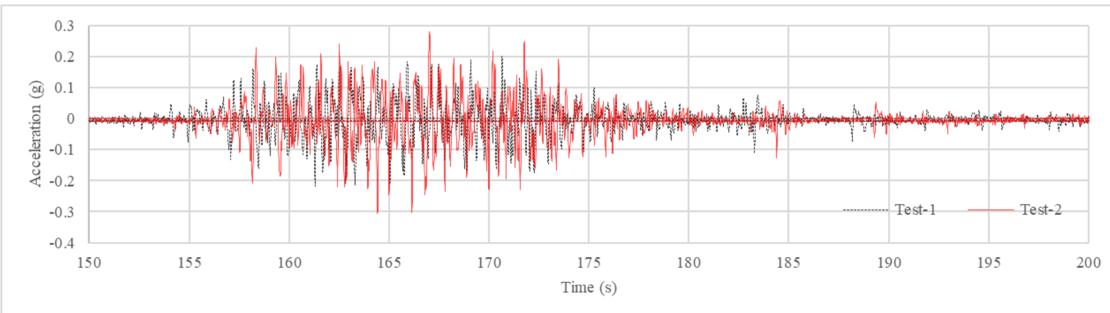

Fig. 2 Input motion used in Test 1 and Test 2

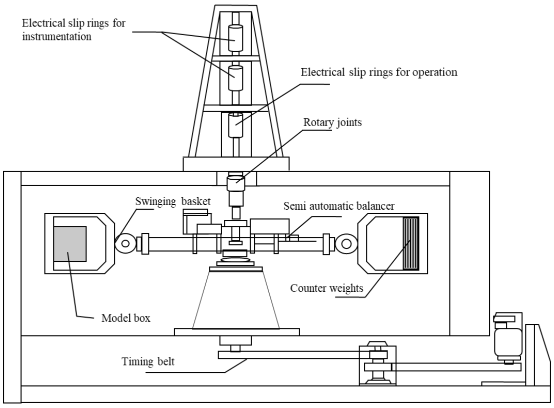

Fig. 3 Simplified schematic diagram of Tokyo Tech Mark III centrifuge machine

3. RESULTS AND DISCUSSION

4.1 Damages of the Embankment Based on Visual Observation

Visual observations of the model were conducted after the centrifuge test. This section discusses the results for both types of model after subjected to earthquake-induced liquefaction.

The conditions of the model reinforced with gravel mat after the centrifuge test is shown in Figure 4(a), and the schematic model condition after centrifuge test is shown in Figure 4(b). The figure shows that lateral spreading occurred indicated by the bends on the contour line at the foundation ground. The embankment undergone settlement shows by the deformation in shape and height difference before and after the test. The toe of the embankment stretch towards the outside and crack was also found at the crest of the embankment.

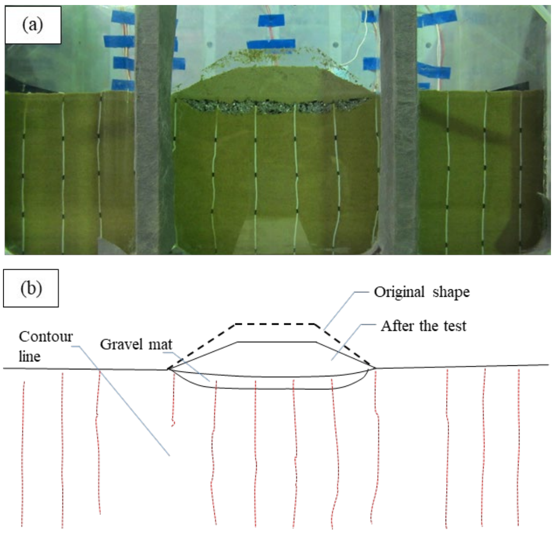

Fig. 4 (a) Condition of the model with gravel mat after the centrifuge test (b) Schematic condition of the model after centrifuge test

Figure 5 shows the conditions of the model with gravel mat and geogrid after the centrifuge test. The embankment undergone settlement and deform in shape indicated by the height difference before and after the test as shown in Figure 5(a). A slight bend on the contour line could be seen from the schematic diagram (Figure 5(b)) indicating that the lateral spreading occurred at the foundation ground.

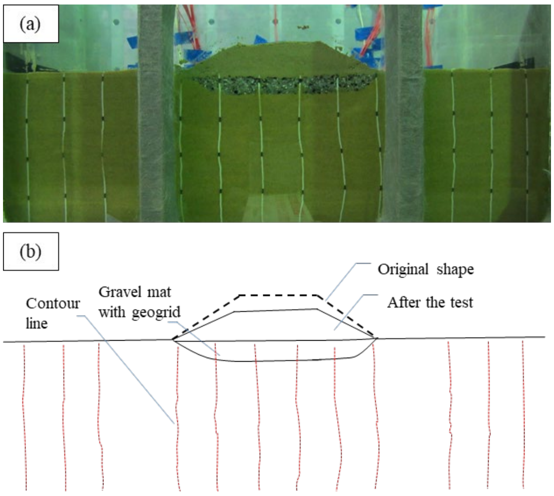

Fig. 5 (a) Condition of the model with gravel mat and geogrid after the centrifuge test (b) Schematic condition of the model after centrifuge test

Based on the test results, both cases show the indication of liquefaction induced damaged to the foundation ground and the embankment. Lateral spreading occurred in the foundation ground as shown by the contour line. The contour line for both of the models beneath the toe and at the free field were bent outward. In some cases, lateral spreading could cause lateral movement over an extensive distance and increase the degree of the damages.

Foundation ground failures were also related to the majority of levee damages in levee performance assessment conducted by Green et al. [11]. The embankment undergone settlement and crack were found at the crest of the embankment. Lateral expansion of the toe of the embankment were also found in both of the cases. The settlement, cracks, and lateral expansion were typical damage and failure patterns of embankment subjected to liquefaction as shown in damaged embankment due to the 2011 Tohoku Earthquake [12].

4.2 Development of Excess Pore Water Pressure

The recorded EPWP for the model with gravel mat is shown in Figure 6(a). The generated EPWP at 6 m depth has a similar value regardless of the position. This condition was also found at the 1.5 m depth, which shows a similar value for each position. The EPWP beneath the toe at 3 m depth developed to the value of 76 kPa higher than the other position, which generates around 30 kPa to 40 kPa of EPWP. This also exceeds the generated EPWP at 6 m depth. In terms of the dissipation rate, the EPWP developed beneath the embankment decrease shortly after it reaches the peak before it becomes stable, which is not the case for EPWP generated at the free field and the toe of the embankment.

The EPWP for the model with gravel mat and geogrid is shown in Figure 6(b). At the 1.5 m and 6 m depth, the generated EPWP has similar value regardless of the placement. The development of the EPWP beneath the toe of the embankment is slightly different with EPWP at 3 m depth become the highest EPWP generated and exceed the generated. There are not much differences shown in terms of dissipation rate during 150-180 s period. However, a slight drop in EPWP is visible at 6 m depth beneath the centre of the embankment EPWP. This was not found at the 3 m and 1.5 m depth.

The generated excess pore pressure for each of location are at similar value as the effective overburden stress generated at the model. It is an indication that the foundation ground of the model has undergone liquefaction.

Figure 7 compares the development of EPWP at the free field and beneath the centre of the embankment at 1.5 m depth. It also shows the input motion for each test. The dynamic load starts to increase rapidly at around 155 s. When the dynamic load starts to increase, it triggers the sudden development of EPWP at the foundation ground. Both proposed countermeasures succeed in minimising the development of EPWP as compared to the previous study [21]. Based on the test results, EPWP generated at the free field almost doubled the EPWP generated at the centre of the embankment at 1.5 m depth. The slight difference shown between the input motion of Test 1 and Test 2 might come from the sensitivity of the sensors installed in the model.

In terms of the dissipation rate, the EPWP beneath the centre of the embankment starts to dissipate at around 210 s of time. This is faster than the dissipation of EPWP at the free field which starts to dissipate at around 270 s. The EPWP will dissipate to a constant residual value at the value

The gravel mat installed beneath the embankment might have helped the process of dissipation as it acts as drainage. Gravel drains are effective in dissipating the excess pore water pressure after shaking [29]. The installation of drainage is proved to be effective in increasing the seismic performance. Drastic improvement in seismic performance of a large scale embankment was found in the study conducted by Enomoto and Sasaki [30], which installed a toe drain to successfully lowering the seepage water elevation.

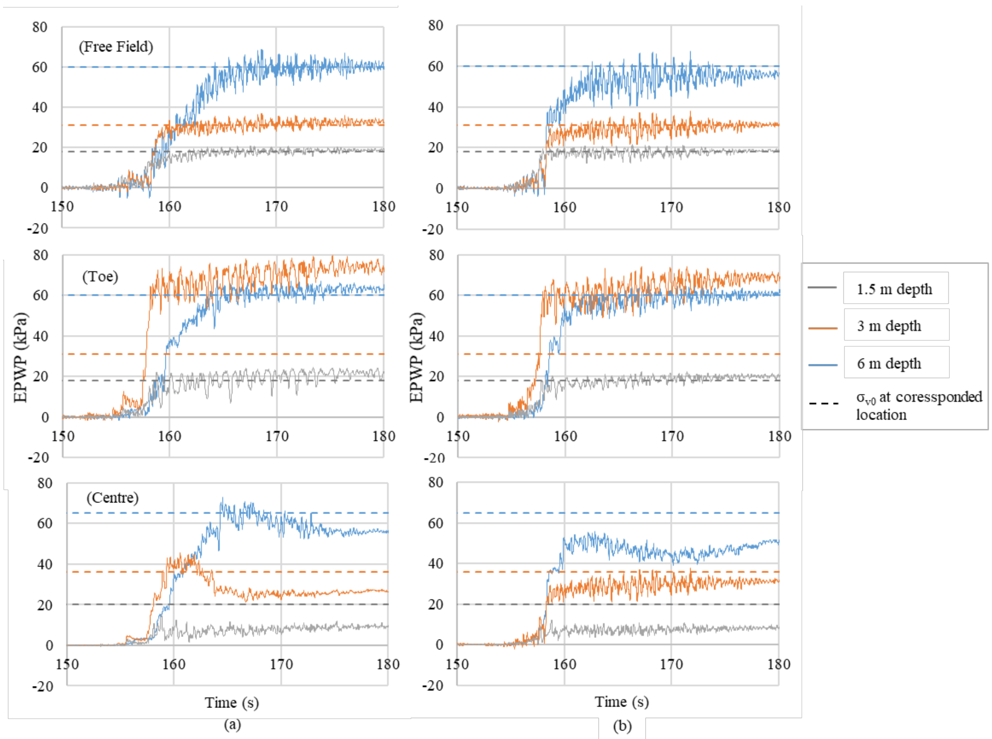

Fig. 6 Excess pore water pressure generated for (a) Model with gravel mat and (b) Model with gravel mat and geogrid at free field, beneath the toe, and beneath the centre of the embankment at 1.5 m, 3 m, and 6 m depth

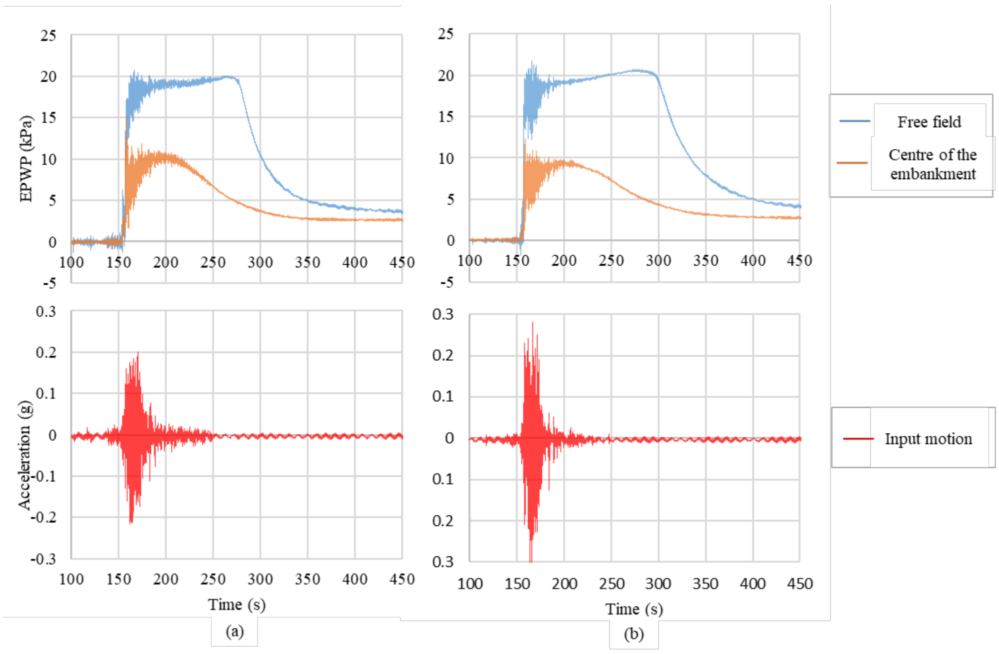

Fig. 7 Excess pore water pressure with its respective input motion for the model with (a) gravel mat and (b) gravel mat with geogrid at 1.5 m depth

4.3 Settlement of the embankment

From the visual observations after the test, the embankment undergone settlement as could be seen in Figure 4 and Figure 5. The installed LDVT recorded the settlement of the embankment for both cases. Based on Figure 8, the maximum settlement for the model with gravel mat is 0.31 m (Figure 8(a)) and for the model with gravel mat and geogrid is 0.26 m (Figure 8(b)). The embankment reinforced with gravel mat and geogrid have less settlement compared to the embankment reinforced with gravel mat.

Albeit the difference is only 0.05 m, the usage of geogrid as a middle layer on the gravel mat slightly improve the seismic performance of the embankment compared to the one that only uses gravel mat as reinforcement. The combination of gravel mat and geogrid might also contribute in reducing the settlement of the foundation ground beneath the embankment and limiting the uplift movement of the embankment during the earthquake-induced liquefaction.

Sasaki et al. [19], in their study based on the conducted shaking table test, found that geogrid could reduce the amount of deformation of an embankment which is in line with the test result observed in this study. However, it also found that the geogrid does not reduce the settlement due to the deformation of the foundation ground. The findings are further validated by means of numerical analysis

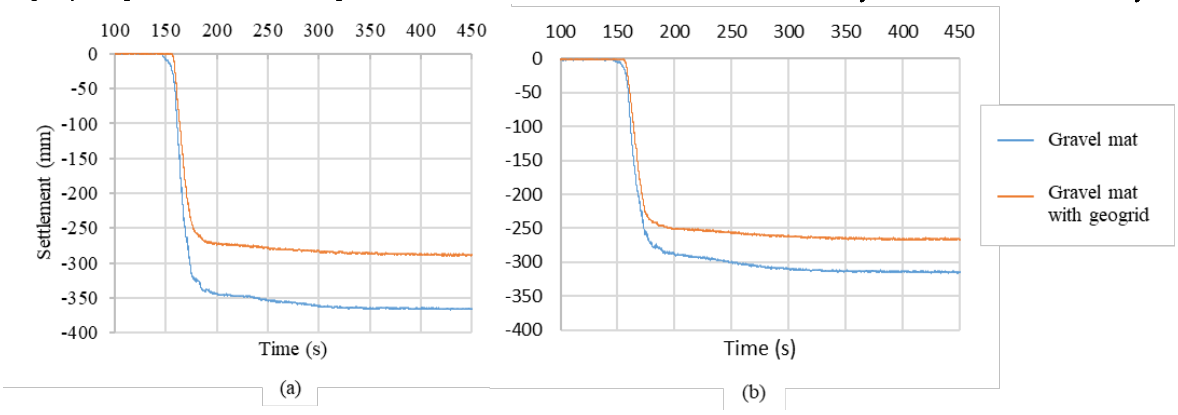

Fig. 8 Settlement at the toe and the crest of the embankment

4. CONCLUSION

Embankment mitigated using gravel mat and gravel mat with geogrid were modelled by means of physical modelling. The model then tested

using centrifuge test to simulate the prototype conditions under defined boundary conditions. Geotechnical centrifuge test enables to replicate the actual conditions of the soil through physical modelling. The objective of this study is to learn the liquefaction mechanism and its consequences and also the effectiveness of the proposed mitigation in mitigating liquefaction-induced damages.

The foundation ground undergone lateral deformation which indicating the occurrence of liquefaction. The rapid dynamic load applied to the model caused sudden development of EPWP. The excess pore pressure then reduces the initial effective overburden stress resulting in loss of strength of the soil. Although the result shows that the EPWP generated at free field and toe is not significantly different. From the result, we can see the reduction in generated EPWP beneath the center of the embankment, thus shows the effect of the proposed countermeasures.

The settlement was observed for both of the embankments. There are no significant differences in the amount of settlement for both embankments with gravel mat and embankment with gravel mat and geogrid. However, the settlement of the embankment with gravel mat and geogrid is less than the embankment with gravel mat which is 0.26 m and 0.31 m respectively. Based on this, the combination of gravel mat and geogrid might have a slightly better impact in reducing the settlement compared to the model without geogrid.

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