Proceedings of the 9ICEG
9th International Congress on Environmental Geotechnics
25-28 June, 2023 | Chania, Greece
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https://doi.org/10.53243/ICEG2023-227

Liquefaction of Mine Waste Rock and Its Mitigation Using Vertical Drains in Centrifuge Model Tests

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ABSTRACT

In this study, dynamic centrifuge tests were performed to evaluate the performance of vertical drains in mitigating liquefaction of mine waste rock. Testing was conducted at the centrifuge acceleration of 50 g to simulate a 8-m unform level profile of loose waste rock representing by a saturated poorly-graded sand with gravels and fines. Each model was subjected to a sinusoidal shaking motion with peak acceleration of 0.4 g in prototype scale for a total 40 number of cycles. Results were analyzed by comparing results of the baseline test (without drains) with the model test with the vertical drains. The study examined a pattern of 18 vertical drains having a prototype diameter of 150 mm installed to a depth of 4 m with a spacing of approximately 1 m. Results show that the top half of the model without drains liquefied based on the development of pore water pressure. For the model with drains, excess pore pressure in the top half of the model reduced by approximately 15-20% and liquefaction did not occur. There was no difference in the excess pore pressure at the deeper depths below the drains for both models. Upward dissipation of pore water was observed at deeper depths for both models due to the relatively high permeability of the soil. Surface settlement occurred as the model was densified in both tests. However, the use of drains to reduce soil settlement was not clearly observed as it is likely due to the presence of gravels.

Keywords: centrifuge modelling, mine waste rock, gravelly sand, liquefaction, earthquake drains

1 INTRODUCTION

Liquefaction of tailings and waste rocks is recognized as a major hazard in mining infrastructure worldwide. Tailings are typically fine-grained material left over from the milling process during the extraction of metals from ore, whereas waste rock is the bedrock that has been mined but does not contain valuable metal of interest. While fine-grained tailings have been known to liquefy when subject to static and dynamic loads, liquefaction susceptibility of waste rocks is challenging. Waste rocks are typically composed of a wide range of particle sizes, including boulders, gravels, sands, and fines. Studies on the effects of soil composition and fabric of relatively large coarse-grained material on soil liquefaction are limited. Despite this, previous research studies have shown that loose gravelly soils are liquefiable, however factors influencing liquefaction and its consequence require more studies (e.g., Ruttithivaphanich and Sasanakul 2022, Basham and Athanasopoulos-Zekkos 2020, Hubler 2017, Andrus 1994).

Several ground improvement techniques have been developed to mitigate liquefaction in sandy soils by reducing the generation of excess pore pressure. Sands mixed with gravels would seemingly have high permeability and high stiffness, hence, they are presumably less susceptible to liquefaction. As a result, ground improvement techniques are not widely applied to this type of material in practice. A limited number of research studies has been conducted to evaluate prefabricated vertical drains (PVDs) and other type of drains (sand and gravel drains) using centrifuge modelling testing (e.g., Hung et al. 2016, Olarte et al. 2017, Howell et al. 2012, Kirkwood and Dashti 2019, Madabuashi and Garcia-Tores 2021).

Centrifuge modelling techniques provide the advantage of simulating realistic field stress condition in a model subjected to a controlled loading condition while the model responses can be observed through dense array of sensors. Past centrifuge studies focused on the use of drains to migrate liquefaction of sandy soils, however no prior centrifuge study has examined similar ground improvement technique for sands mixed with gravels and fines. As mentioned above, the reason is that liquefaction behaviour of gravelly soils is not well studied. This study aims to investigate liquefaction behaviours of waste rock and the potential use of vertical drains to mitigate liquefaction. The paper presents results of two centrifuge tests prepared with the same soil and subjected to the same dynamic loading conditions. The first test was studied without the drains and served as the baseline test. Additional test was conducted to investigate the potential use of vertical drains to mitigate liquefaction.

2 CENTRIFUGE MODEL PREPARATION AND PROCEDURE

2.1 Mine waste rock sample

The material tested in this study was waste rock sample collected from a mine site. Dry and wet sieve analysis was performed to develop a grain size distribution curve as shown in Figure 1. The composition of the sample is 40% gravel, 48% sand, and 12% non-plastic fines. According to the unified soil classification system the material is classified as poorly graded sand with silt and gravel (SP-SM). Based on visual classification, the gravel and sand particles are subangular. This is to be expected as the mine waste rock is essentially any rock that is leftover after the ore is removed and is gone through the crushing and milling process to reduce particle size. To simulate a loose deposition of the waste rock, the sample was tested at a target dry density in a range of 17.6 kN/m^3 which is equivalent to a void ratio of approximately 0.53.

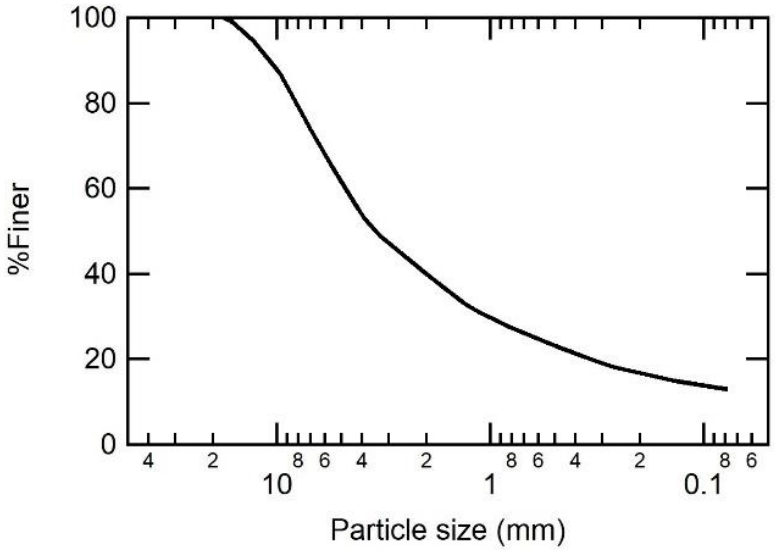

Figure 1. *Grain size distribution of mine waste rock sample*

2.2 Design of earthquake drains

Model of earthquake drains were designed to simulate prototype drains typically made of perforated plastic pipes wrapped with geotextile fabric. In this study, the model drains were made of hollow Santoprene thermoplastic tubing with an inside diameter of 3 mm to simulate a 150 mm diameter drains in prototype. The type of material was previously used in Olarte et al (2017) and Howell et al (2012) to simulate prefabricated vertical drains (PVDs). The tube was cut to the desired length and arrays of holes were made throughout the length of the tube. Filter paper was used to wrap around the drains to minimize clogging. The drains were installed in a triangle pattern with a spacing of approximately 1 m and depth of 5.7 m in prototype. With this design, the area ratio between the drains and treated soil is approximately 2% which is equivalent to the common drain design area ratio of 1-2% and spacing of 1-3.5 m in field applications (Howell et al 2012, Basu and Prezzi 2007, Holtz 1987).

2.3 Centrifuge model

Two centrifuge model tests conducted at 50 times earth gravity (50 g) were presented in this paper. Waste rock sample was prepared at the same initial conditions for both tests following the same procedure, as presented below.

The centrifuge model was prepared by moist tamping the waste rock to achieve the dry unit weight of 17.6 kN/m^3. The moist tamping method was necessary as it minimizes the segregation of soil particles, thus produce a uniform mixture. The model container shown in Figure 2(a) is a laminar container having dimensions of 203 mm in width, 279 mm in length, and 203 mm in height. The sample was placed in the container in 4 layers to reach an initial model height of 152 mm. Pore water pressure sensors and accelerometers were placed between these layers as shown in Figure 3. Linear vertical differential transducers (LVDTs) were installed at the surface of the model to measure settlement, and additional accelerometers were installed on the side the model container. The model was then saturated with a viscous fluid of 50 mPa·s (at 20 °c).

For the model with drains, the model drains were installed after the bottom layer was completed. The drains were hold in place during the subsequent layers. A long piece of wood was temporarily inserted in each drain to minimize the lateral deformation during the model preparation. Sample was carefully compacted near the drains. It is noted that because of the sample was prepared at loose state, the compaction effort was minimal, hence the non-uniformity of the model around the drains was not a concern.

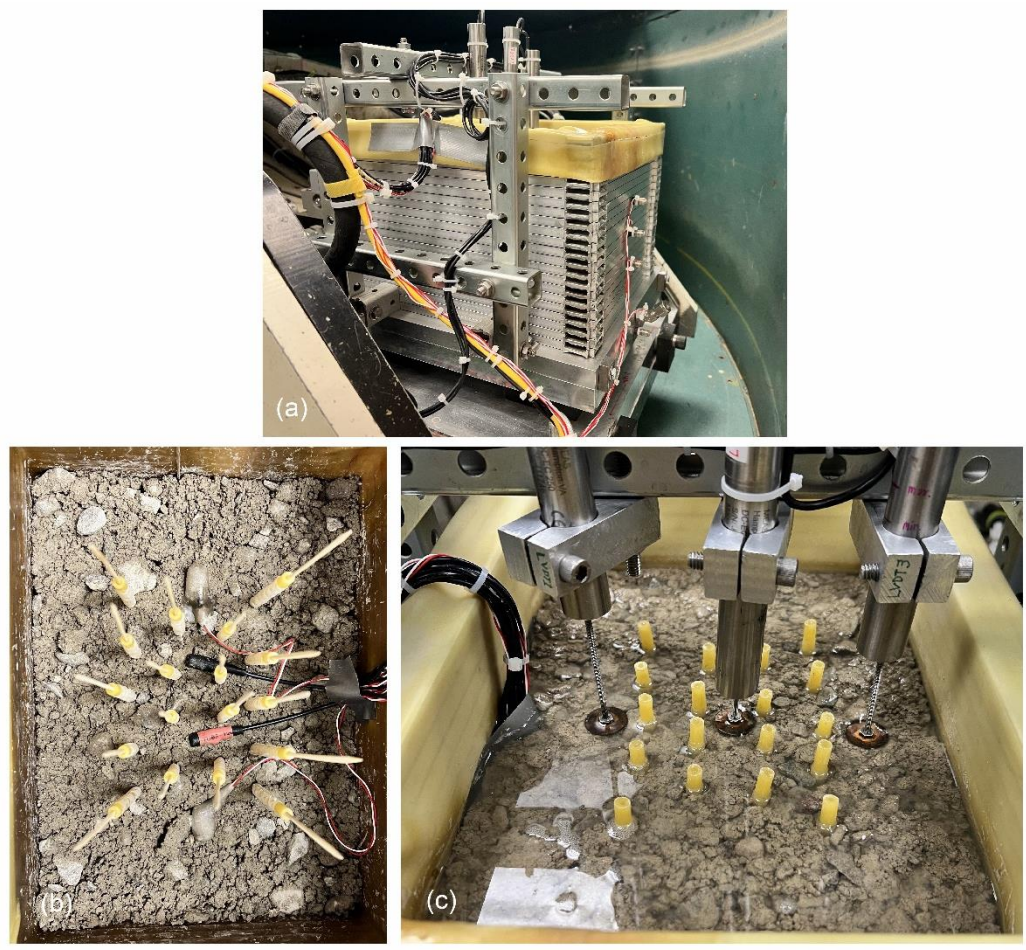

Figure 2. *Photographs of centrifuge test setup a) centrifuge laminar container and shaker b) top view of the model with drains during model preparation, b) surface of the model with drains prior to centrifuge test*

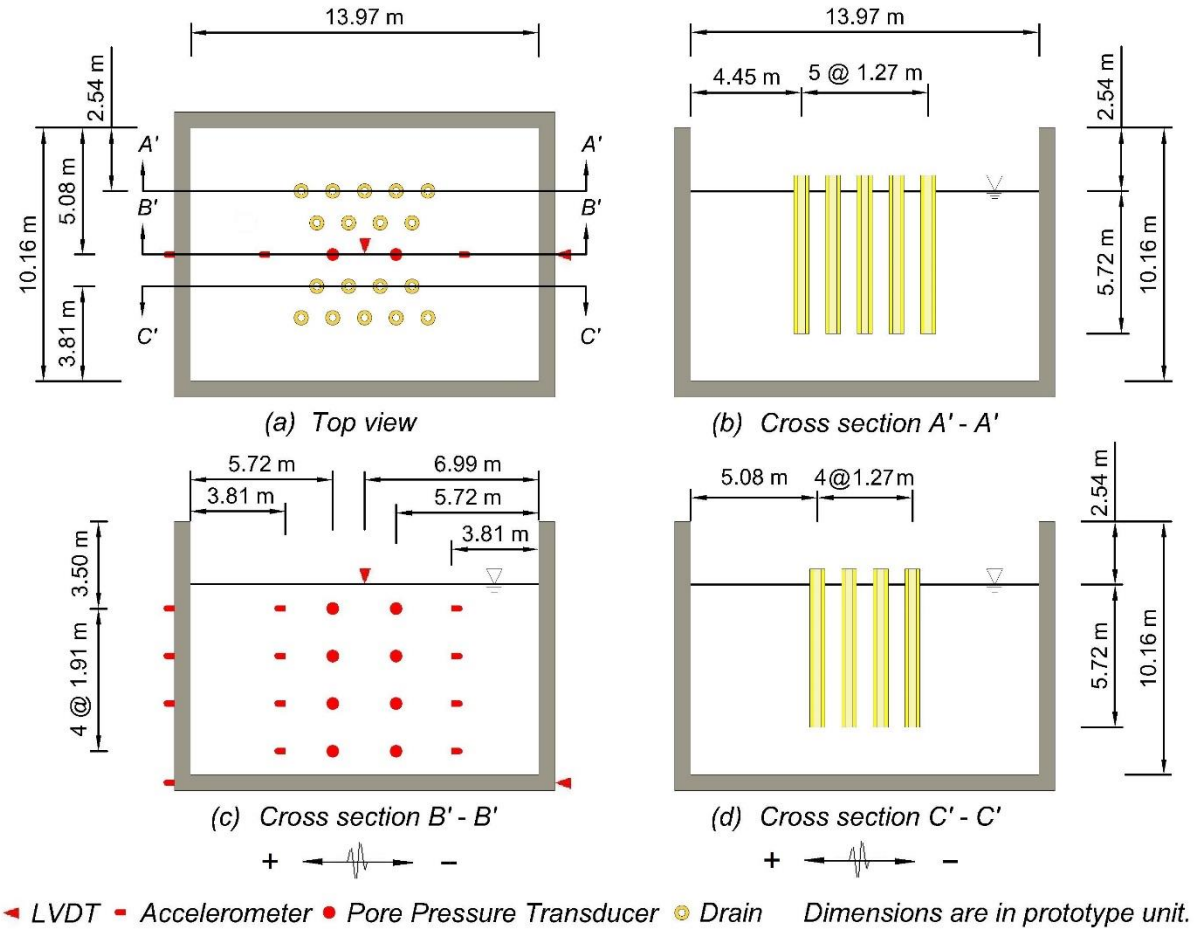

Figure 3. *Sensor and drain locations in a centrifuge model*

2.4 Testing procedure

The centrifuge model was spun to 50 g to consolidate under one-way drainage conditions. Once the excess pore water pressure was dissipated; the model was then shaken using a 1-Hz uniform sinusoidal motion with an amplitude of about 0.4 g for 40 cycles generated by the centrifuge shaker. After the shaking completed, the model was held at 50 g for additional time until the excess pore water pressure was dissipated. The test was completed after the centrifuge stopped. Post-test model dissection was conducted to manually observe model displacement and final sensor locations.

3 RESULTS

3.1 Consolidation at 50 g

Prior to shaking, both models were consolidated at 50 g for approximately 5 minutes. Dissipation of excess pore water pressure was observed very quickly due to the high permeability of the models. Settlement was observed during the centrifuge spinning up and at 50 g resulting in the increase in model unit weight from 17.6 kN/m^3 to about 18 kN/m^3. The void ratio decreased from 0.53 to 0.48. Both models had the same initial condition prior to shaking.

3.2 Dynamic Testing

3.2.1 Shaking motions

The shaking motion applied to the base of the model using the centrifuge shaker is shown in Figure 4 for each test. The Arias intensity was observed to be 21.81 and 21.55 m/s for the model without drains

and with drains, respectively. The average amplitude for both motions is approximately 0.41 g. The observation verified that both models were subjected to the same shaking motion.

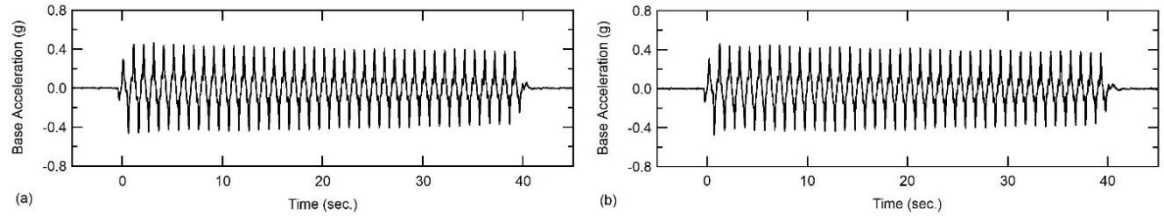

Figure 4. *Base acceleration for (a) model without drains, (b) model with drains*

3.2.2 Acceleration time histories

Acceleration time histories measured along the soil profile for both models are presented in Figure 5. at the model depths of 1, 2.6, 4.8, and 6.7 m, corresponded to the effective vertical stress of approximately 10, 30, 50, and 70 kPa, respectively. It is noted that the positive acceleration represents the initial cyclic shear direction while the negative acceleration represents the reverse cyclic shear direction. Based on the figure, asymmetrical acceleration time histories were clearly observed in both models at 1, 2.6, and 4.8 m. While at a depth 6.7 m, the acceleration appears to be uniform with an average amplitude of about 0.34 g. As the model subjected to the positive cyclic shear direction, the amplitude of acceleration reduced with the number of cycles. Reduction in acceleration relates to the reduction in shear stress which implies the loss in soil strength. As the model subjected to the reverse cyclic shear direction, the negative spikes were observed resulting in asymmetrical response in acceleration time histories. These behaviours were observed in both tests and have been reported in previous centrifuge modelling studies (e.g., Manandhar et al. 2021, Ruttithivaphanich and Sasanakul 2023) of dense sand and gravelly soils as an indication of soil dilatancy. This implies that the waste rock mobilized cyclic resistance under the cyclic shear reversal. In addition, collision between gravel particles could possibly cause the spikes when the model was under the reverse cyclic shear direction. The waste rock tested in this study is loose gravelly sand composed of 40% gravel; hence the gravel behaviour likely contributes to the dynamic response of the material. Comparison between the two tests, it appears that the amplitude of negative spikes in acceleration in the model with drains is relatively higher than that of the model without drains, particularly at the top half of the model where the drains were located.

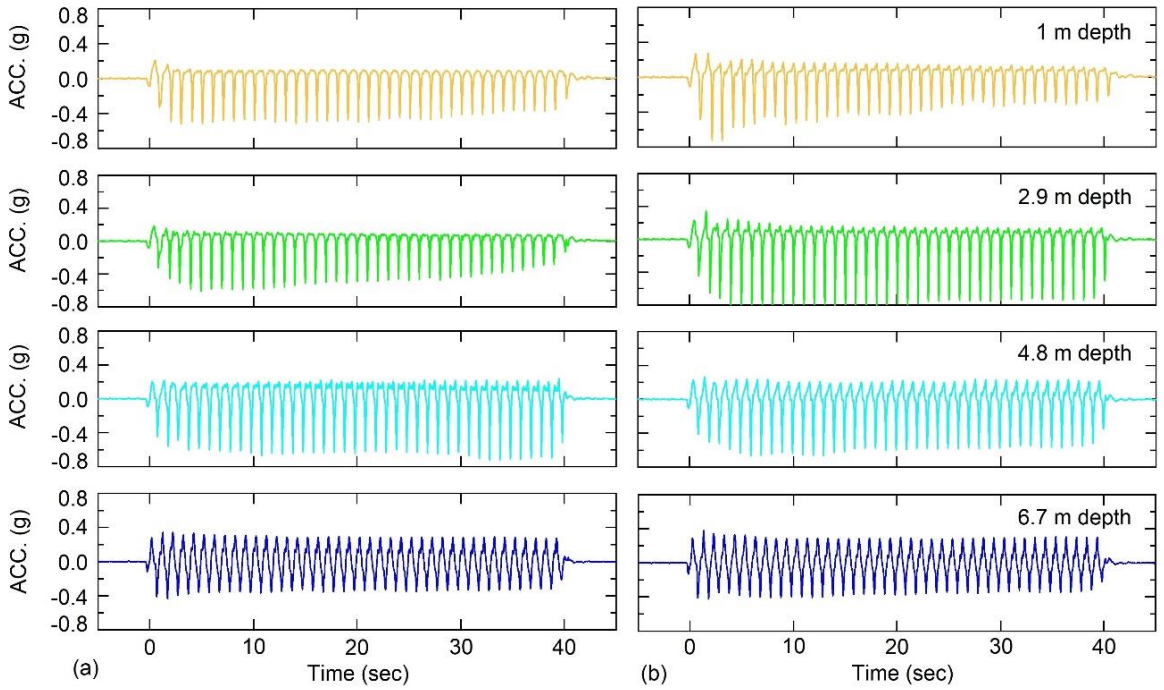

Figure 5. *Acceleration time histories for (a) model without drains and (b) model with drains*

3.2.3 Pore water pressure development

Development of excess pore water pressure during shaking measured along the soil profile of the two tests are presented in Figure 6. For both models, excess pore water pressure was observed for the entire model depth, hence the gravelly sand has the potential to liquefy and lose its strength and stiffness under cyclic loading. Strong oscillation with high amplitude of negative spikes were also found in the records of excess pore water pressure. It is likely related to the soil dilatancy and mobilization of cyclic resistance under the cyclic stress reversal while the excess pore water pressure accumulates and continues to increase. This observation was observed at all depths for both models. Interestingly, at the depth of 6.7 m, the dissipation of excess pore water pressure was observed during the shaking event, and it is likely due to the upward dissipation of pore water pressure. As mentioned, the acceleration time histories observed at this depth remained fairly uniform indicating that the soil did not significantly lose its strength and stiffness at this depth. The highest excess pore pressure development appears at the depth 4.8 m. For the model with drains, this location corresponds to the bottom of the drains. Less excess pore pressure for the model with drains was observed than that of the model without drains. At the depth 4.8 m, the strongest oscillation with high negative amplitude was observed. It is possibly related to the flow of water through the bottom and holes along the side of the drains. At the surface of the model, dissipation of pore water pressure is clearly observed for the model with drains while the excess pore pressure continued to develop for the model without drains.

Liquefaction is assessed by using the value of pore water pressure ratio, R_u (a ratio between excess pore pressure, ΔU, and initial vertical effective stress). R_u value of 1 means the effective stress in soil is zero hence there is no contact between soil particles. R_u value of more than 1 has been reported in centrifuge modelling tests particularly the locations near the surface of the models due to several factors including uncertainties in sensor locations, additional stress due to model movements, etc. Regardless, when R_u value of more than 1 was found in a centrifuge model, liquefaction certainly occurred as the soil experiences significant amount of excess pore water pressure. Based on the observed pore pressure development behavior, the R_u value of 0.90 is used as a criterion for the initiation of soil liquefaction. As shown in Figure 7, the model without drains liquefied from the surface to a depth of 4.6 m as the R_u value exceeded 0.9. Within the liquefied layer, the pore pressure developed to R_u of 0.5 within the first cycle of shaking. Soil liquefaction occurred at the cycle number 2, 6, and 9 at the depth of 1, 2.9, and 4.8 m, respectively. Pore water pressure remain high throughout the 40 shaking cycles although some dissipation of excess pore water pressure was observed during the last few cycles of shaking. It is clear that the drains are effectively reduced the excess pore water pressure. The R_u values for the model with drains remained below 0.90. It is interesting to observe that at the depth 4.6 m, the R_u approached the maximum value of 0.89 at about 15 cycles but dissipated quickly after that. In addition, at the depth 6.7 m, the R_u values reached the maximum value of about 0.59 for both model tests. Overall, with the design of the drains having the area ratio between the drains and treated soil of about 2%, liquefaction of gravelly sands was successfully mitigated by reducing the development of excess pore water pressure and increasing the rate of dissipation. For this testing configuration, the reduction of excess pore pressure was approximately 15-20%.

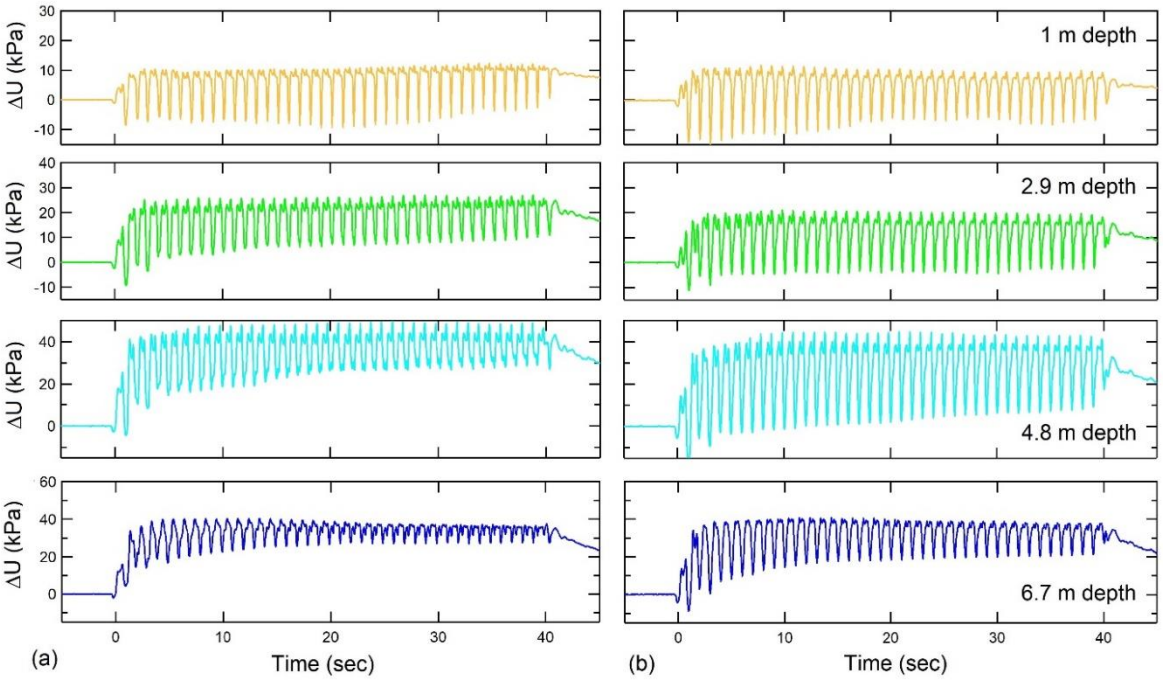

Figure 6. *Excess pore pressure for (a) model without drains and (b) model with drains*

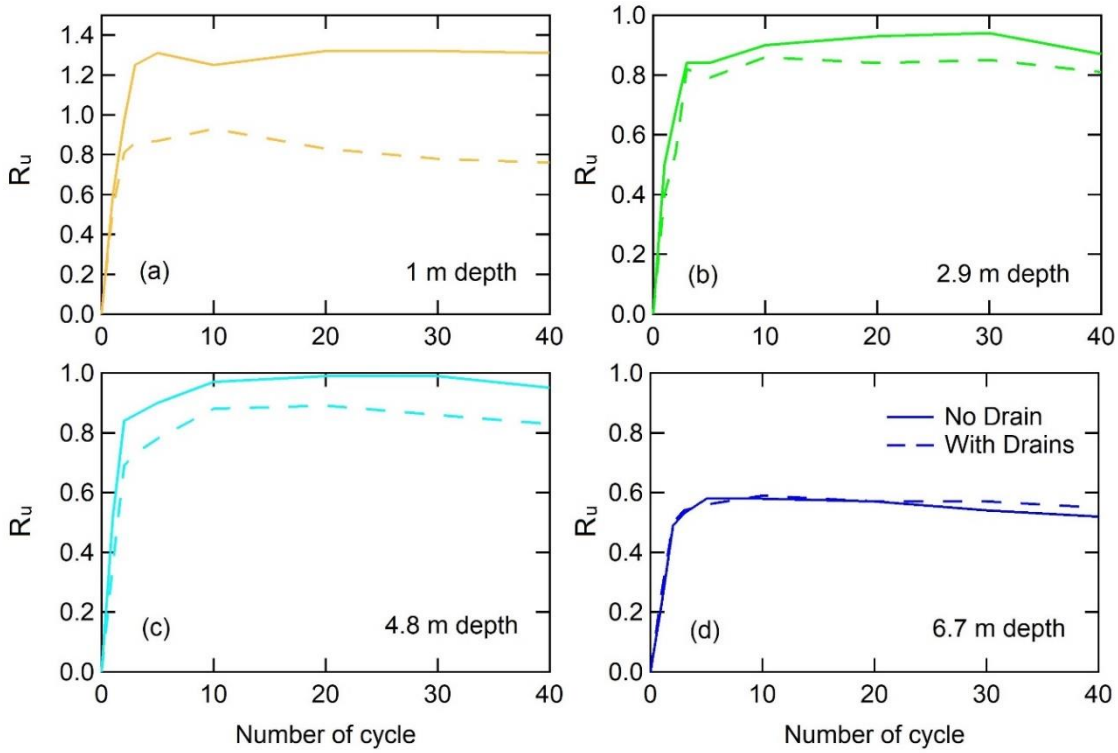

Figure 7. *Pore water pressure ratio with number of cycles for (a) model without drains and (b) model with drains*

3.2.4 Surface settlement

Post-test investigation was conducted after the completion of the test. This included the measurement of sensor locations after test which is used to determine soil settlement, and to calculate the change of void ratio along the depth of the model. For both model tests, it was found that there was a decrease in void ratio toward the bottom of the models. In the comparison to the initial void ratio of 0.48 estimated

prior to shaking, the void ratio for both models was approximately 0.3, 0.4, and 0.2 at the depth of 2.9, 4.8, and 6.7 m, respectively. Due to the high uncertainty in surface measurement (top 40 mm of the model or about 2 m in prototype), the exact value of void ratio could not be accurately determined. The surface of the models was found to be relatively loose due to the fines migration and suspension in the fluid at the surface of the model as shown in Figure 8. Careful measurements of 9 locations showed that the surface settlement was not uniform. Relatively higher settlement was observed away from the drains or near the container wall, and lower settlement was observed at the area near the drains or at the center of the model. Values of surface settlement due to shaking for both models appears to be similar as they are ranging from 0.4-0.6 m in prototype (8-12 mm in the model scale). As a result, the use of drains to improve settlement was not clearly observed. It is possible that due to a high amount of gravel content for these tests (40% gravel), the gravel particles densified, and the particle interlocking occurred after the shaking in both tests. Even though the excess pore pressure was reduced by the vertical drains, it was not enough to provide a noticeable improvement of soil settlement after liquefaction. This observed behaviour is likely influenced by the mixing ratio between gravels, sands, and fines.

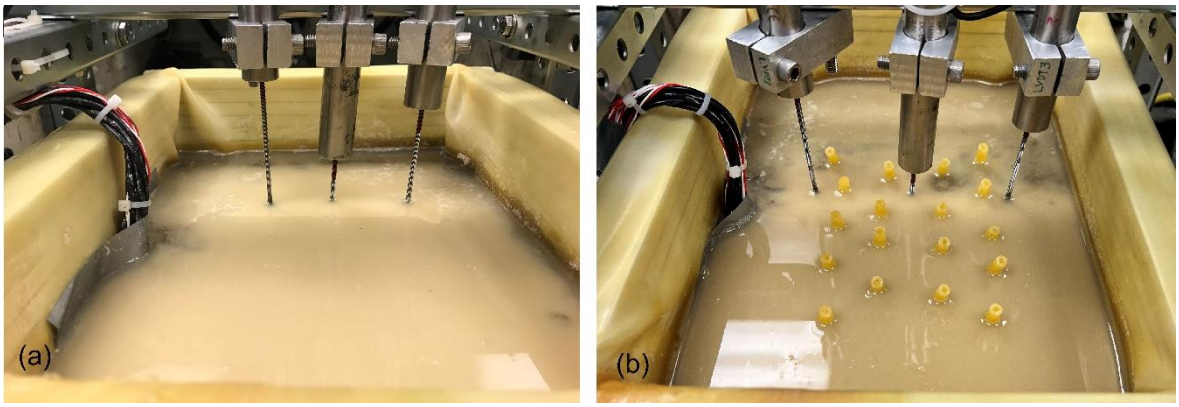

Figure 8. *Visual observation of surface settlement after testing (a) model without drains and (b) model with drains*

4 CONCLUSIONS

This study investigates liquefaction behaviours and its mitigation of mine waste rock composed of sands, gravels, and fines. Two centrifuge tests were conducted to assess the effectiveness of the earthquake vertical drains. The model of waste rock without drains liquefied under that the 1-Hz uniform sinusoidal shaking with the amplitude of 0.41 g. Pore water pressure ratio reached the value of 0.90 and higher from the surface to a depth of 4.6 m. For the model with drains subjected to the same loading condition, no liquefaction was observed as the pore pressure ratio was reduced to values less than 0.90. Reduction of excess pore pressure was approximately 15-20%. For both models, dilation spikes in acceleration time histories and oscillation of pore water pressure indicates that the waste rock tends to regain strength and stiffness during cyclic loading. The use of drains helps accelerate this process. Dissipation of the pore water pressure was observed to be faster in the model with drains. Therefore, the findings of this study can be concluded that the vertical drains mitigate soil liquefaction as they reduce the excess pore water pressure development and accelerate the pore water dissipation. This may be considered a promising application of vertical drains in mine waste rock. However, based on a limited number of tests in this study, the use of vertical drains to reduce settlement is not significant. Future research is needed to evaluate the influence of gravels on surface settlement to evaluate the effectiveness of the earthquake drains in sands with gravels and fines.

5 ACKNOWLEDGEMENTS

This research was partially funded by the Geosynthetic Institute for the GSI Fellowship Program for the second author. This financial support is greatly appreciated. Any opinions, findings, and conclusions included in this paper are those of the authors and do not necessarily reflect the viewer of the funding agency.

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