An Investigation of the Cyclic Behaviour of Tailings Using Shaking Table Tests: Effect of a Drainage Inclusion on Porewater Development

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ABSTRACT
When subjected to cyclic loadings, such as those induced by earthquakes, hard rock mine tailings may be subject to liquefaction, which can cause failure of the retaining dikes. This paper presents the results of a laboratory investigation that focussed on the dynamic behaviour of tailings due to the cyclic loads on a shaking table. Different scenarios were investigated to assess the influence of various factors including tailings density and rigid and/or drainage inclusions Results obtained on tailings, with and without a drainage inclusion, are presented and discussed here.

RÉSUMÉ
Lorsque soumis à des chargement cycliques, tels ceux produits par un tremblement de terre, les résidus de mines en roches dures sont susceptibles à une liquéfaction, qui pourrait engendrer la rupture des digues de retenue. Ce projet de recherche porte sur l'analyse en laboratoire des changements physiques produits lorsque des résidus miniers sont soumis à des sollicitations cycliques sur table vibrante. Différents scénarios ont été étudiés sur le modèle physique afin d'évaluer l'effet de plusieurs facteurs, incluant la densité initiale des résidus et l'insertion d'inclusions rigides et/ou drainantes. Les résultats obtenus sur les résidus miniers, avec et sans inclusion, sont présentés et discutés dans cet article.

1 INTRODUCTION

Dams forming tailings impoundments are some of the most impressive structures built by humans. They can contain hundreds of millions of cubic meters of tailings. In the event of a dam breach, the consequences to downstream infrastructures and populations can be catastrophic and the economic and environmental damage can be significant. In this regard, one of the greatest challenges for geotechnical engineers is to insure the seismic stability of these structures, during operation and after mine closure (Vick 1990; Aubertin et al. 2002a, b).

It is estimated that there are more than 3,500 appreciable tailings dams worldwide (Davies 2002). ICOLD (2001) classified 219 past events of tailings dam failure and indicated that the three major causes were, in order, slope instability, earthquakes, and water over-topping. Earthquakes affect the stability of embankments and dams in two main ways: by the application of horizontal loads and through the development of excess porewater pressures leading to strength loss and liquefaction (e.g. Kramer 1996). Liquefaction is the almost complete loss of strength in loose, saturated, cohesionless materials due to excess porewater pressure development caused by dynamic loading applied with such rapidity that the excess porewater pressures cannot dissipate during loading. Tailings from hard rock mines can generally be classified as very loose, saturated, non-plastic sandy silts and are thus vulnerable to liquefaction (e.g. Vick 1990; Aubertin and Chapuis 1991).

There have been many laboratory studies on the dynamic behaviour of naturally occurring soils, such as sands and silty sands (e.g. de Alba and al. 1975; Walker and Stewart 1989; Boulanger and Idriss 2006). However, there have been relatively few studies on the dynamic behaviour of tailings (e.g. Mittal and Morgenstern 1977; Ishihara et al. 1981; Garga and McKay 1984). More recently, Wijewickreme et al. (2005) and James (2009) extended our knowledge on the behaviour of tailings when subjected to dynamic loadings.

Stone (gravel) columns are commonly used to control the effects of liquefaction in sandy soils deposits by providing additional stiffness and drainage to reduce or prevent the development of excess porewater pressures during dynamic loading (e.g. Sasaki and Taniguchi 1982; Barksdale 1987)). The use of stone columns in silt deposits was studied by Adalier et al. (2003), who found that the primary effect was to increase the stiffness of the soil mass during cyclic loadings, leading to a reduction in the shear strains and thus the excess porewater pressure development. Aubertin et al. (2002a) and James (2009) explored the use of waste rock inclusions to control the effects of liquefaction in tailings impoundments. The research described in this paper used shaking table testing to evaluate the dynamic behaviour of tailings and the effect of rigid and/or drainage inclusions. Tests with this physical model were also conducted on sand for comparative purposes.

2 THE PHYSICAL MODEL

2.1 Configuration

The model is composed of a box with a rigid aluminum frame. It is 1-m-square, with a height of 75 cm. As shown on Figure 1, one side of the box is made of a 19-mm-thick sheet of acrylic for observation of the material during testing. The details of the box design have been presented elsewhere (Jolette 2002; James et al. 2003).

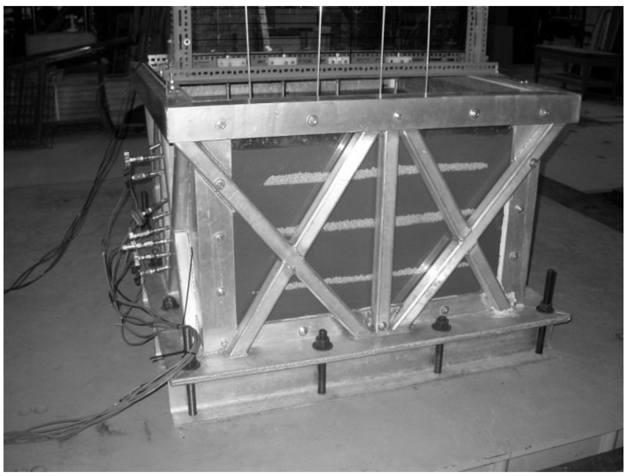

Figure 1. The instrumented box used for the tests on the shaking table.

The experimental program discussed here was conducted on sand and hard rock tailings. The results of the tests on the sand were used to assess the testing system response with respect to shaking table tests done by others and for comparison with the results on tailings. A total of 11 tests were conducted, 3 on sand and 8 on tailings. Three of the tests on tailings were conducted without inclusions and 5 were conducted with inclusions.

The inclusions consisted of a coarse sand that was used to form a flexible column, a wall and/or a base layer in the model, or of a slotted, 10-cm-diameter PVC tube, filled with coarse sand or tailings, to create a rigid column in the center of the model. The rigid column was attached to the bottom and to the top of the instrumented box.

The inclusions composed of coarse sand were placed to provide drainage for the relief of excess porewater pressures generated during shaking. The rigid column served to reinforce the tailings sample and possibly decrease the generation of excess porewater pressures by reducing the shear strains of the sample during shaking. The rigid column filled with coarse sand was place to provide both drainage and reinforcement. Earlier studies have been made on those types of inclusions (drainage in sands or reinforcement in silts) by Sasaki and Taniguchi (1982), Barksdale (1987), Adalier et al. (2003) and Martin et al. (2004). However, these studies used naturally occurring soils and their applicability to tailings is unclear.

2.2 Material Properties

The tested tailings were made of a combination of milling wastes from two hard rock mines from the Abitibi region in the province of Quebec. The result was a silty mixture with very low plasticity. The grain size distribution is shown on Figure 2, along with the range of gradation for materials most susceptible to liquefaction based on Hunt (1986). Generally, naturally occurring soils of similar gradation as the tailings are somewhat more plastic and thus less susceptible to liquefaction (Lee and Fitton 1968; Kramer 1996). Nonetheless, tailings with no to low plasticity are expected to be highly susceptible to liquefaction (Mittal and Morgenstern 1977; Aubertin and Chapuis 1991).

Testing on sand was used to verify that the results of the physical modeling were in agreement with published results from similar testing on sand and to provide a benchmark for comparison with the tests results on tailings. The sand used was a mixture of a silty sand and coarse sand which formed a silty sand (see Figure 2).

Laboratory tests were performed on small samples of the tailings and of the sands to define their basic properties. These are summarized on Table 1.

Table 1. Summary of materials properties.

Characteristics	Tailings	Silty sand	Coarse sand
USCS classification	ML	SM	SP
Solid grain relative density	3,385	2,756	2,67
Maximum dry unit weight (kN/m³)	22,41	17,95	N.A.
Hydraulic conductivity k_{sat} (cm/s)	$2.2\text{x}10^{-05}$	$1.3\text{x}10^{-03}$	$1.4\text{x}10^{-02}$
Void ratio (e) for k_{sat}	0,62	0,69	0,60
Grain size D_{10} (mm)	0,0051	0,037	0,21
Grain size D_{50} (mm)	0,04	0,14	0,40
Grain size D_{60} (mm)	0,048	0,18	0,42

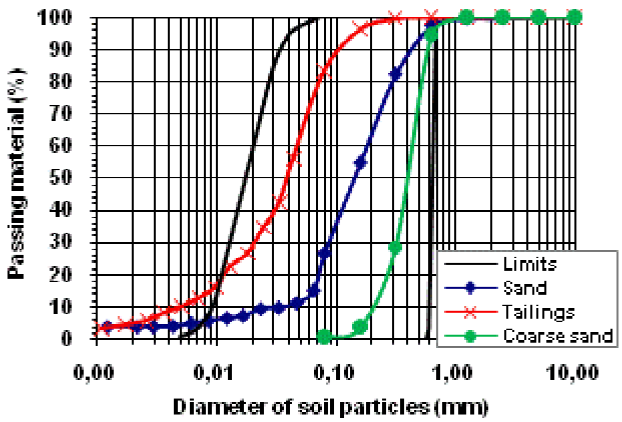

Figure 2. Grain size curves of the materials used and limits of liquefiable soils based on laboratory testing (from Hunt 1986).

2.3 Sample Preparation

After field sampling, the tailings were kept under water to prevent oxidation which could change their physical properties. The following procedure was used to prepare the tailings samples for the shaking table tests: a) the saturated tailings were placed in the box with a shovel in five 10-cm-thick layers and the agglomerations were broken with a scoop; and b) the size of each layer was measured and samples were taken to determine the void ratio, e (an average value of 0.65 was measured). One of the main goals of the sample preparation was to keep this value as uniform and constant as possible. The final sample height was 50 cm, on average.

For the sand samples, dry sand was poured through a 2 mm sieve from the top of the box to break (or retain) any agglomerations. Each 5-cm-thick layer was slightly densified with a metal plate to a mean void ratio of 0.90. A total of 12 layers, for a total height of 60 cm, was used. The sample was then saturated from the bottom under a hydraulic gradient of 2.

In some tests, a coarse sand was added to the model to form a drainage inclusion. In Tests 6 and 8, the coarse sand was poured dry into a centered slotted PVC tube. The tube was removed in Test 8 to create a drainage column but was retained (with the sand) in the tailings in Test 6 to create a rigid/drainage inclusion. For Tests 9 and 11 the coarse sand was placed between two metal plates installed in the center of the model and was slightly compacted with a small shovel. The thin plates (separated by 10 cm) extended the width and height of the box to separate the tailings into two parts. Before shaking, the two plates were removed, leaving a 10-cm-thick wall of coarse sand in the center of the tailings.

After the sample was prepared, it was left in-place for 3 days (on average), prior to testing. The water level in the sample box was kept slightly above the surface to prevent desaturation. Just before shaking, excess water was removed from the surface.

2.4 Instrumentation

During sample preparation, various instruments were placed at different levels in the model. Figure 3 is an isometric schematic view of the instrumented model.

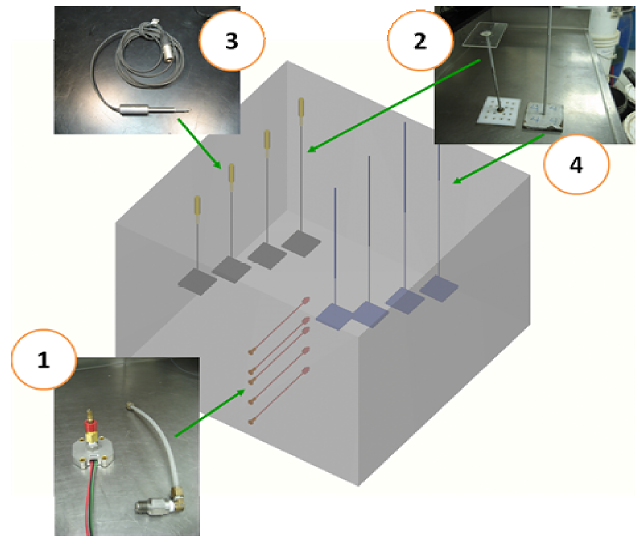

Figure 3. Schematic view of the instrumented model.

Each model tested was instrumented with pressure sensors (item 1 of Figure 3) to measure the water pressure at various depths. The sensors were linked to the outside of the box to tailings using plastic tubes with filter protection to prevent the infiltration of particles. Porewater pressures were measured near the center of the box when there were no inclusions or 15 cm from the inclusions. During shaking, porewater pressures were registered every 0.1 seconds. The pressure transducers used are the PX240A series by Omega with a pressure range of ±5psi (±35 kPa).

Displacement sensors (LVDTs) were installed (item 3 of Figure 3) to monitor the vertical movement of the tailings at different depths. They were attached to thin metal tubes connected to lightweight, perforated plastic plates (item 2 of Figure 3). The plastic plates were 10-cm-square, 1-cm-thick and were installed at different depths. The plates were perforated to limit displacement due to seepage pressures. The miniature displacement transducers used were LVDT model SC3 DC-DC by Honeywell with a range of ±0.2 inch (±0.51 cm).

Heavier plates were also placed at various depths. These were the same size as the plastic plates, but they were made of steel (item 4 of Figure 3). Small diameter metal tubes were screwed into the center of the metal plates and extended 30 cm above the surface of the sample. A graduated plastic board was installed behind these tubes to follow the movement of the metal plates. Due to their weight the metal plates applied stresses of 0.75 kPa to the samples and it was expected that their movement could be used to monitor the change in bearing capacity due to excess porewater pressure development and liquefaction.

Finally, thin layers of coloured sand were placed at various depths in the sample to observe and measure the deformation during and after the dynamic loading of the models.

2.5 The Shaking Table

The shaking table used was located in the Structural Engineering Laboratory at Ecole Polytechnique in Montreal. It is a high performance seismic simulator with one dimension of movement (horizontal), a 3.4-m-square testing platform, a bearing capacity of 15 tons, a maximal displacement of 125 mm, a maximal speed of 800 mm/s, and a maximum acceleration of 3 g. The system can reproduce real or digitally created earthquake signals or produce harmonic and random motion.

3 SHAKING TABLE TESTING

As mentioned above, 11 shaking table tests were conducted for this program, 3 on the sand and 8 on tailings (with and without inclusions). All the tests were conducted using a sinusoidal loading with a frequency of 1 Hz. The tests on sand were conducted with peak horizontal accelerations (PGA) of 0.12 g, 0.17 g and 0.35 g, corresponding to horizontal displacement amplitudes of 25, 37.5 and 75 mm. These were used, in part, to determine the dynamic loading condition that would induce significant excess porewater pressures in the models. The tests on tailings were conducted with at a peak horizontal acceleration of 0.12 g (horizontal displacement amplitude of 25 mm). The loading cycles lasted from 5 to 20 minutes (300 to 1,200 cycles) depending of the material response. Monitoring of the models was continued for 16 to 24 hours after the end of the dynamic loading to observe the post-loading response, specifically the dissipation of excess porewater pressures and any further deformation. The tests conducted are summarized on Table 2.

The dynamic loadings used for this project were not meant to be representative of actual earthquake ground motion. The signal used was only one-dimensional, of uniform amplitude, of constant frequency, and of long duration (many cycles). However, the use of these loading conditions allowed for good observation of the dynamic behaviour of the materials and relative comparisons of their responses. These types of loadings are often used in liquefaction research (Ishihara 1996).

Table 2. Summary of shaking table tests.

Test	Material	Inclusion	PGA
1	Sand	-	0.12g
2	Sand	-	0.17g
3	Sand	-	0.35g
4	Tailings	-	0.12g
5	Tailings	-	0.12g
6	Tailings	Rigid column & sand	0.12g
7	Tailings	Rigid column	0.12g
8	Tailings	Sand column	0.12g
9	Tailings	Sand wall	0.12g
10	Tailings	-	0.12g
11	Tailings	Sand wall & bed	0.12g

As indicated previously, the three tests on sand (Tests 1, 2 and 3) were conducted to evaluate the response of the physical model on the seismic table for different cyclic loading conditions and for comparison with published shaking table test results. These results will not be presented here; details can be found in Pépin (2009). Tests 4, 5 and 10 were conducted using tailings without inclusions and were used as reference tests. Tests 6 and 7 used tailings with the slotted 10-cm-diameter PVC tube placed vertically at the center of the model. In test 6, the column was filled with coarse sand to provide drainage. In Test 7, the column was filled with tailings to isolate the influence of the rigid column. To isolate the influence of the coarse sand, Test 8 was conducted with a 10-cm-diameter column of sand in the middle of the tailings sample (no tube). Tests 9 and 11 used tailings with a 10-cm-wide sand wall in the center of model, orientated perpendicularly to the direction of loading. Test 11 consisted of tailings with a 10-cm-wide wall of coarse sand in the center of the model and a 10-cm-thick layer of sand at the bottom of the model. Results of Tests 5 and 9 are presented below; others will be presented elsewhere (see Pépin 2009).

4 SELECTED TESTS RESULTS

Different testing scenarios were evaluated. One of the objectives was to assess the influence of the inclusions on the dynamic behaviour of the tailings. The main results of only two tests are reported in this article (due to space limitations).

The dynamic response of the tailings is evaluated here using Test 5 (tailings without inclusion) and Test 9 (tailings with a 10-cm-thick wall of coarse sand in the middle). The evaluation focuses on the results obtained by the pressure transducers that were used to measure the porewater pressure in the models during testing.

4.1 Excess porewater pressure development

The excess porewater pressure at various depths in the models was calculated by subtracting the initial pore water pressures registered 45 s prior to loading from the measured pore water pressures.

The cyclic loading for Test 5 consisted of 1,000 cycles (or 1,000 s because the loading frequency is 1 Hz) of loading with a peak acceleration of 0.12 g. The response of the model was monitored during loading and for 60,000 s (16.7 h) afterwards. The excess porewater pressure development at various depths during loading and for 1,000 s afterwards can be seen in Figure 4. It can be observed that at the start of shaking, the excess porewater pressures at different depths increased at different rates. The maximum values were reached within 100 to 800 s of the start of shaking, from the top of the sample downwards. Once the maximum values were reached, the excess porewater pressure stabilized or diminished gradually for the rest of the shaking period. This phenomenon was also observed in the other tests and may be explained by a dilative response of the material to loadings at very low effective stresses.

Immediately after the end of shaking, there was a very rapid increase in the excess porewater pressures throughout the depth of sample. This increase can be attributed to the cessation of the cyclic loading which allowed the tailings to contract, creating additional porewater pressures. After such rapid increases in the excess porewater pressures at the end of shaking, the excess porewater pressures were relatively stable for the next 1,000 s (see figure 4).

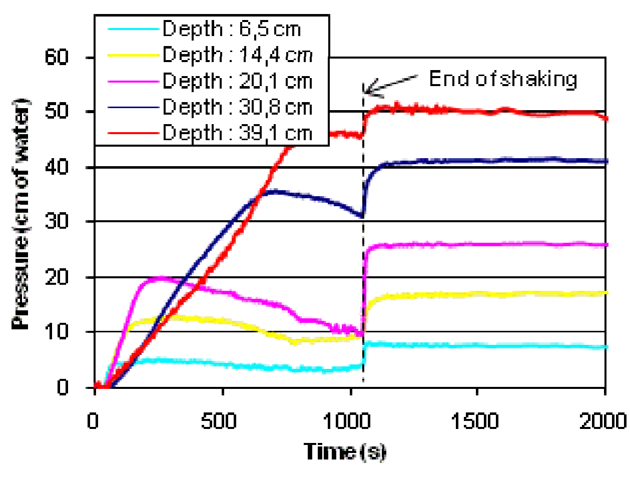

Figure 4. Test 5: Development of excess porewater pressures Δu in the tailings at various depths (0 to 2,000 s).

Figure 5 shows the excess porewater pressure development for Test 5 from the start of shaking to 16.7 h afterwards. It can be seen that after the end of shaking, the excess porewater pressure declined to near zero within 50 000 s (14 h) and were then relatively stable, indicating a return to hydrostatic conditions. However, at some depths, the excess porewater values declined to less than zero. This behaviour may be attributed to some excess pore pressures that existed in the model at same location (especially at depth) before the start of shaking. Additional analysis is underway to fully assess the tailings behaviours during this (and other) tests.

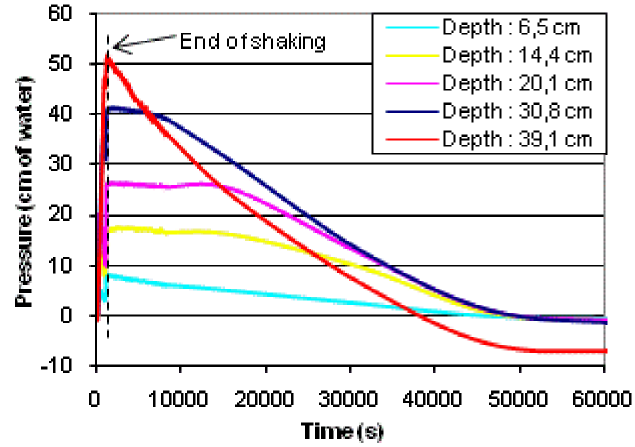

Figure 5. Test 5: Development of excess porewater pressures Δu in the tailings at various depths (0 to 60,000 s).

4.2 Effective Vertical Stress

The Terzaghi equation was used to calculate the effective vertical stresses σ'_v (kPa) at various depths in the model:

$$\sigma'_v = \sigma_v - u \qquad [1]$$

where:

σ_v is the total stress; and

u is the porewater pressure.

The total stress was calculated analytically from the total unit weight ($\gamma_{tailing}$ = 24.17 kN/m^3) and depth (h in m) of the tailings. The porewater pressure was calculated by adding the excess porewater pressure Δu (kPa) (see Section 4.1) to the static porewater pressure (calculated analytically and verified by measurements). Equation [1] can thus be transformed into Equation [2].

$$\sigma'_v = (\gamma_{tailing} \times h) - [(\gamma_{water} \times h) + \Delta u] \qquad [2]$$

Figure 6 shows the evolution of effective vertical stresses from cycles 0 to 2,000. These stresses decreased from the beginning to minimum values and then increased when the excess pore water pressures decreased (Figure 4). At the end of shaking, the effective stresses reached almost zero at every depth. This is due to the rapid increase in the excess porewater pressures due to contraction of the tailings.

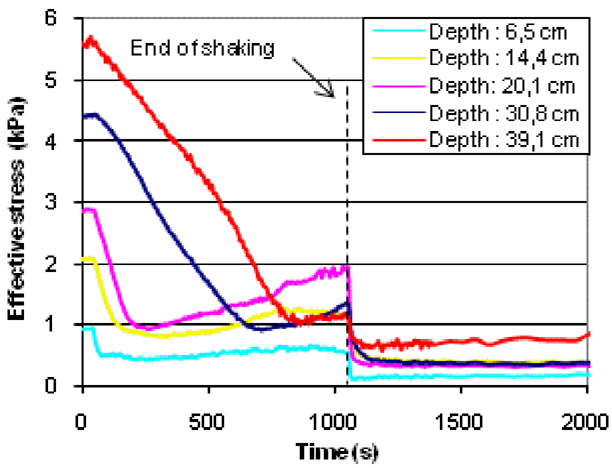

Figure 6. Test 5: Evolution of effective vertical stresses σ'$_v$ in the tailings at various depths (0 to 2,000 s).

Figure 7 presents the evolution of the effective vertical stresses σ'$_v$ at various depths in the model during Test 5. As noted, these stresses are related to the development and dissipation of the excess porewater pressures. As can be seen, the values of σ'$_v$ returned near the initial static condition within 50 000 s (14 h) of shaking.

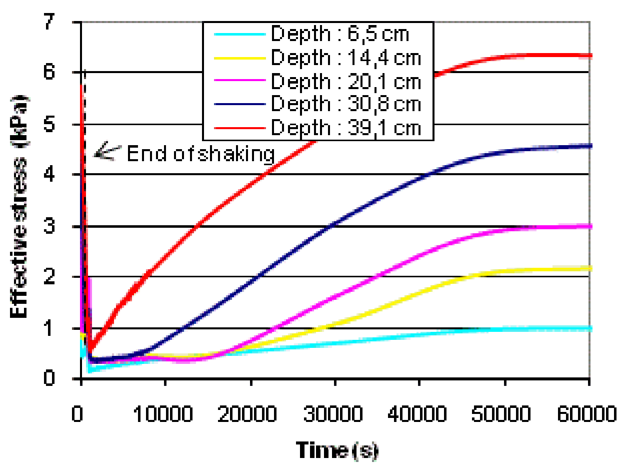

Figure 7. Test 5: Evolution of effective vertical stresses σ'$_v$ in the tailings at various depths (0 to 60,000 s).

4.3 The effect of inclusions

In general, the inclusions introduced in the physical model resulted in a reduction in the maximum excess porewater pressures and a decrease in the rate of excess porewater pressure development. The effects of rigid columns (Tests 6 and 7), drainage columns (Tests 6 and 8) and sand walls (9 and 11) were more pronounced with depth. The results indicate that the inclusion of a wall and bottom layer of coarse sand (Test 11) was the most effective at reducing the development of excess porewater pressure in the tailings. The sand wall inclusion (Test 9) was also very effective, followed by the rigid/drainage column (Test 6). The test with a rigid/non-drainage column (Test 7) seemed to be effective only in the bottom part of the model (from 30 to 50 cm in depth), and had the least effect on excess porewater pressure development.

To illustrate this effect, we will consider the development of the excess porewater pressure Δu in Test 9. This test was conducted on tailings with a 10-cm-thick coarse sand wall located at the center of the model, perpendicular to the direction of motion. Figure 8 shows the variation of Δu for Tests 9 and 5 at depths of 20 cm and 40 cm. It can be seen that there is a significant increase in the time needed to reach the maximum value of Δu with the sand wall (1,050 s instead of 200 at a depth of 20 cm). Also, for Test 9 at a depth of 40 cm, the maximum Δu obtain during shaking was 20 cm of water (2 kPa), which is considerably lower than the value of 46 cm attained at this depth for Test 5.

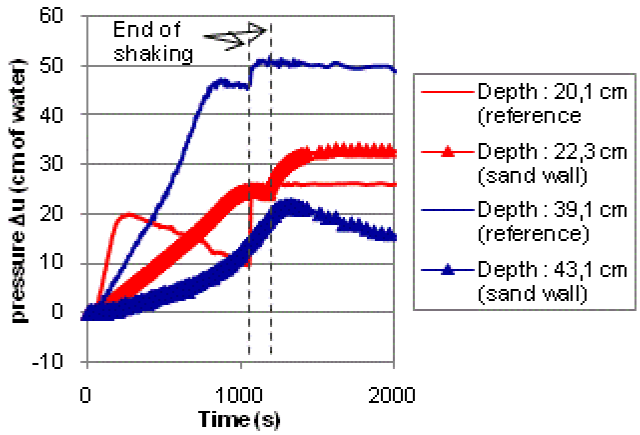

Figure 8. Comparison of the excess porewater pressure development in Test 5 (tailings) and Test 9 (tailings with sand wall) from 0 to 2,000 s.

Figure 9 shows the evolution of excess porewater pressures Δu at various depths during Test 5 and 9 from the beginning of shaking to stabilisation of porewater pressures. A significant difference in the dissipation of pressures in Test 9 is induced by the inclusion of a sand wall, which created a preferential path to evacuate pore pressures. The porewater pressures were totally stabilized after 30,000 s instead of 60,000 s for Test 5.

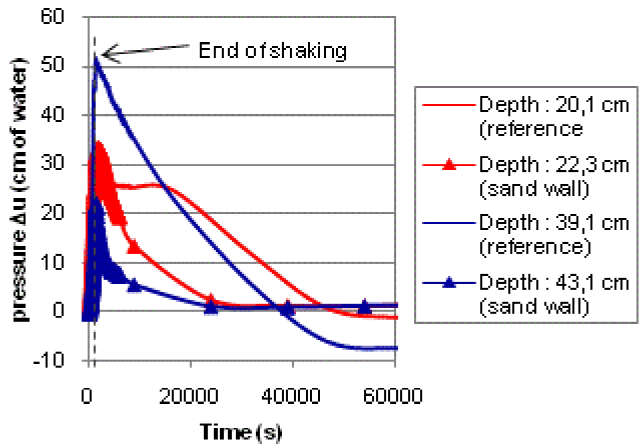

Figure 9. Comparison of the excess porewater pressures in Test 5 (tailings) and Test 9 (tailings with sand wall) from 0 to 60,000 s.

5 DISCUSSION AND CONCLUSIONS

Shaking table testing was conducted on samples of sand and of hard rock mine tailings, with and without inclusions. The inclusions consisted of rigid and/or draining columns, walls and a base layer of coarse sand. A PGA of 0.12 g was sufficient to cause the development of high excess porewater pressures in the tailings without inclusions, but was not sufficient to cause the development of significant excess porewater pressures in the sand. The placement of inclusions in the tailings to provide drainage or mechanical reinforcement, led to an appreciable decrease in the development of excess porewater pressures.

The results of shaking table testing on hard rock mine tailings with and without inclusions and on sand led to the following conclusions:

- Hard rock tailings are susceptible to liquefaction and their liquefaction resistance appears to be somewhat less than that of sand. However, the difference may be due, in part, of the different methods of sample preparation and the different void ratios used for the two materials.
- The presence of rigid and drainage inclusions decreased the rate of excess porewater pressure development in the tailings.
- In all tests, there was a tendency for maximum excess porewater pressures to be first reached at the top and progress downwards to the bottom of the sample.
- It is possible to reduce the liquefaction potential of hard rock mine tailings by installing drainage to inhibit the development of excess porewater pressures and to a lesser extent, by reinforcing the tailings with rigid inclusions. In this series of tests, the installation of drainage was clearly more effective than reinforcement.

Rigid and/or drainage inclusions tested in this study can take the form of ridges or columns of waste rock in a tailings impoundment. It is probable that the results in the impoundment would be fairly similar to those obtained in this study because of the higher permeability and higher shear strength of the waste rock relative to tailings. The effect of waste rock inclusions has been numerically evaluated by James (2009).

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REFERENCES

Adalier, K., Elgamal A., Meneses J. and Baez J.I. 2003. Stone Columns as Liquefaction Countermeasure in Non-plastic Silty Soils, *Soil Dynamics and Earthquake Engineering*, 23: 571-584.

Aubertin, M. and Chapuis, R.P. 1991. Critères de conception pour les ouvrages de retenue des résidus miniers dans la région de l'Abitibi. *Proceedings of the First Canadian Conference on Environmental Geotechnics*, Montréal, Qc, Canada, 113-127.

Aubertin, M., Bussière B. and Bernier L. 2002b. *Environnement et gestion des rejets miniers* [CD-ROM], Les Presses Internationales de Polytechnique, Montréal, Qc, Canada.

Aubertin, M., Mbonimpa, M., Jolette, D., Bussière, B., Chapuis, R.P., James, M. and Riffon, O. 2002a. Stabilité géotechnique des ouvrages de retenue pour les résidus miniers : Problèmes persistants et méthodes de contrôle. Proceedings of Symposium 2002 on Mines and the Environment, CIM, Montréal, Qc, Canada, 526-552.

Barksdale, R.D. 1987. *Application of the State of the Art of Stone Columns – Liquefaction, Local Bearing Failure, and Example Calculations*, *Technical report No. REMR-GT-7*, Washington DC, USA.

Boulanger, R.W. and Idriss, I.M. 2006. Liquefaction Susceptibility Criteria for Silts and Clays, *Journal of Geotechnical and Geoenvironmental Engineering*, 132: 1413-1426.

Davies, M.P. 2002. Tailings Impoundment Failures: Are Geotechnical Engineers Listening? *Geotechnical News*, 31-36.

DeAlba, P., Chan, C.K. and Seed H.B. 1975. Determination of Soil Liquefaction Characteristics by Large-scale Laboratory Tests, *Earthquake Engineering Research Center, report No. EERC 75-14,* Berkeley, CA, USA.

Garga, V. and McKay, L.D. 1984. Cyclic Triaxial Strength of Mine Tailings. *Journal of Geotechnical Engineering*, 110(8): 1091-1105.

Hunt, R.E. 1986. *Geotechnical Engineering Analysis and Evaluation*, McGraw-Hill Inc., New-York, NY, USA.

International Commission on Large Dams & United Nations Environmental Program. 2001. Tailings dams - Risk of Dangerous Occurrences - Lessons Learnt from Practical Experiences, *Commission Internationale des Grands Barrages*, 121, Paris, France.

Ishihara, K., Sodekawa, M. and Tanaka, Y. 1981. Cyclic Strength of Undisturbed Mine Tailings. *Proceedings of the International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*. St-Louis, Missouri, USA, pp. 53-58.

Ishihara, K. 1996. *Soil Behavior in Earthquake Geotechnics*. Oxford University Press, Oxford, NY, USA.

James, M. 2009. *The Use of Waste Rock Inclusions to Control the Effects of Liquefaction in Tailings Impoundments*, Ph.D Thesis, École Polytechnique de Montréal, Qc, Canada.

James, M., Jolette, D., Aubertin, M. and Bussière, B. 2003. An Experimental Set-up to Investigate Tailings Liquefaction and Control Measures. *Proceedings of the International Symposium on Major Challenges in Tailings Dams,* ICOLD, Montreal, Qc, Canada, pp. 153-164.

Jolette, D. 2002. Élaboration d'un système sur simulateur sismique afin d'étudier le potentiel de liquéfaction des sols. Projet de fin d'étude (unpublished report), École Polytechnique de Montréal, Qc, Canada.

Kramer, S.L. 1996. *Geotechnical Earthquake Engineering*, Prentice Hall Inc., Englewood Cliffs, NJ, USA.

Lee, K.L. and Fitton, J.A. 1968. Factors Affecting the Cyclic Loading Strength of Soil, *Vibration Effects of Earthquakes on Soils and Foundations*, ASTM STP 450: 71-95.

Martin, J.R., Olgun, C.G., Mitchell, J.K. and Durgunoglu, H.T. 2004. High-modulus Columns for Liquefaction Mitigation, *Journal of Geotechnical and Geoenvironmental Engineering*, 130(6): 561-571.

Mittal, H. and Morgenstern, N.R. 1977. Design and Performance of Tailings Dams. *Proceedings of Conference, Geotechnical Practice for Disposal of Solid Waste Material.* ASCE, Ann Arbor, MI, USA, 475-492.

Pepin, N. 2009. *An Investigation of the Cyclic Behavior of Tailings Using Shaking Table Tests.* Master Thesis, École Polytechnique de Montréal, Qc, Canada (To be submitted in 2009).

Riemer, M., Moriwaki, Y. and Obermeyer, J. 2008. Effect of High Confining Stresses on Static and Cyclic Strengths of Mine Tailing Materials, *Geotechnical Earthquake Engineering and Soil Dynamics IV Congress*, ASCE, Sacramento, CA, USA, pp. 1-10.

Sasaki, Y. and Taniguchi, E. 1982. Shaking Table Tests on Gravel Drains to Prevent Liquefaction of Sand Deposits, *Japanese Society of Soil Mechanics and Foundation Engineering – Soils and Foundations*, 22(3): 1-14.

Vick, S.G. 1990. *Planning, Design and Analysis of Tailings Dams*, BiTech publishers Ltd., Vancouver, BC, Canada.

Walker, A.J. and Stewart, H.E. 1989. Cyclic Undrained Behaviour of Nonplastic and Low Plasticity Silts, *National Center for Earthquake Engineering Research, report NCEER-89-0035*, Buffalo, NY, USA.

Wijewickreme, D., Sanin M.V. and Greenaway, G. 2005. Cyclic Shear Response of Fine-grained Mine Tailings, *Canadian Geotechnical Journal*, 42: 1408-1421.