PHYSICAL MODELLING OF LANDSLIDES IN LOOSE GRANULAR SOILS

by

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Abstract

The catastrophic consequences associated with landslides necessitate predictions of these hazards to be made with as much certainty as possible. However, the often complex nature of these events make predictions highly challenging. In this thesis, a number of hypotheses related to the triggering mechanisms and subsequent consequences of landslides in a loose-granular soil were investigated. The investigation was conducted using small-scale geotechnical centrifuge models, and a new flume facility developed to examine landslide behavior in a reduced-scale model.

The first hypothesis explored in this research was that static liquefaction might preferentially occur in the saturated granular soil located at the base of the landslide rather than the well-drained inclined portion of the slope. Using a geotechnical centrifuge model, it was found that a small initial toe failure did act as a monotonic loading trigger to shear the loose contractile saturated sand at the base of the slope and caused liquefaction to occur.

The second hypothesis investigated whether the consequences of a landslide triggered under elevated groundwater antecedent conditions are higher than scenarios under drier antecedent conditions. Results from five centrifuge models subjected to different antecedent groundwater conditions show that higher groundwater conditions can result in landslides with velocities about three times higher and travel distances eight times higher than low antecedent conditions.

The third hypothesis investigated the influence of slope inclination on landslide consequences. Seven geotechnical centrifuge models were built and tested, comparing the consequences of landslides triggered in 20° and 30° sloped models with different groundwater conditions. The results of these tests found that the influence of slope angle on the mobility consequences of a triggered landslide are highly dependent on the antecedent groundwater conditions. The most
significant case was under high groundwater conditions, where the shallower 20° slope travelled twice the distance and speed of the steeper 30° slope.

A new flume facility was developed to examine landslide behaviour in a reduced-scale model, and a direct comparison was made to one of the centrifuge models from the research. The comparison demonstrated the challenges associated with using reduced-scale models to study suction-dominated problems such as hydraulically-induced landslides in loose granular soils.
Co-Authorship

This thesis presents primarily original work by the author, however the author wishes to acknowledge the significant contributions made by the geotechnical centrifuge modelling team and by co-authors who collaborated on the research topic of antecedent groundwater conditions leading to a journal paper written by the author and used as Chapter 3 in this thesis. Author order of submitted manuscripts reflects an equal sharing of first authorship between the author and her supervisor.

Chapters 1 and 6 are entirely the original work of the author.

Chapter 2 is prepared from work that has been published in the *Canadian Geotechnical Journal*:


Chapter 3 is prepared from work accepted for publication in the Journal *Landslides*:


Chapter 4 has been adapted from a draft manuscript entitled:

Chapter 5 has been adapted from a draft manuscript entitled:


The geotechnical centrifuge tests were conducted in St. John's Newfoundland by the C-CORE modelling team, led by former Queen's research assistant Jeff Kemp.
To my grandfather, Donald Blaine Redfern,
if I can be half the Civil Engineer he was, I will have accomplished great things in my life

and to Paige Just Lauren,
for her unwavering love and support. I couldn’t have done this without you.
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To the Staff and Faculty of Civil Engineering
Thank you for all your help, support, smiles, and guidance throughout my (long, long) time in Ellis. You have put up with much from me, and I will always be exceedingly grateful.

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Much of this work is thanks to the blood, sweat and tears you left behind in that flume. This would not have been possible without you. Thank you for spending a summer "living in my world".

To my family and friends
Thank you for everything. Everything! For your unwavering love, support, time on the basketball court, messages of well wishes, nights of fun, and for leaving me be when I asked you to. But most of all, for believing that 12 years, 4 months and 14 days is an acceptable length of time to be a student at Queen's.

To Paige
This thesis is but a small token of what my PhD has brought me. For it brought me to you. Thank you for sharing this 'chapter' with me, and for being willing to share with me in the rest of life's adventures.

To Andy
Well, it's official. The journey is over. But oh what a journey it has been! Thank you. Andy, you have shown a bottomless supply of patience, advice, support, and time, and I will always be indebted to you (and Kelly, Ollie and Ella too) for all you have done for me. You have been the difference maker in my professional life, and I thank you for having been willing to take the chance on me. I consider myself lucky to be able to call you not only my professor, supervisor, mentor, and colleague, but also my friend. Thank you!

Alrighty. I think it's time to start talking and thinking about landslides and physical models, and as Andy likes to say "……but what does it all mean?"
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<tbody>
<tr>
<td>$C_c$</td>
<td>Coefficient of curvature</td>
</tr>
<tr>
<td>$C_u$</td>
<td>Coefficient of uniformity</td>
</tr>
<tr>
<td>$D_{10}$</td>
<td>Effective particle size</td>
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<tr>
<td>$D_{50}$</td>
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<tr>
<td>$e_{\text{min}}$</td>
<td>Void ratio, minimum</td>
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<tr>
<td>$g$</td>
<td>Gravitational acceleration</td>
</tr>
<tr>
<td>$G_s$</td>
<td>Specific gravity of soil particles</td>
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<tr>
<td>$h$</td>
<td>Thickness of soil layer</td>
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<td>$k$</td>
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<td>Porewater pressure ratio</td>
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<td>$\tau$</td>
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Chapter 1

Introduction

1.1 General

In 1972, after an unusually heavy period of rainfall, a devastating landslide occurred in Hong Kong. The volume of the landslide was greater than 40,000 m³ which collided with 2 large buildings and killed 67 people. In 1976, another large landslide occurred, killing 18 people, and bringing the total landslide fatalities to over 175 in a four-year period. In response to these tragedies the government of Hong Kong formed the Independent Review Panel on Fill Slopes which began to try and understand these fast-moving, rainfall-induced landslides that had impacted their city.

Jumping ahead to the present day, the geotechnical engineering community is still trying to understand the behaviour of these rapid moving hydraulically-induced landslides. A task which becomes more complex as changes to our climate have made periods of heavy rainfall more frequent, which in turn increases the number of landslides they trigger (Crozier, 2010). These increased periods of heavy precipitation (e.g. Trenberth, 2010) generate additional volume of free flowing water that seeps into the soil, creating a loss of suction beneficial to stability. This in turn decreases the effective stress in the slope's soil, and there is a reduction in the stability of the slope (e.g. Wang and Sassa, 2007; Moriwaki et al., 2004; Take et al., 2004). This type of triggering mechanism can initiate landslides of all types, however the failure mechanism most often attributed to the triggering of high mobility landslides (and therefore high risk landslides) in these and other loose granular soil is static liquefaction (e.g. Knill et al., 1976).
1.2 Static Liquefaction

Static liquefaction (liquefaction triggered from monotonic loading rather than cyclic motion such as an earthquake) is a form of deviatoric strain-softening in which an imposed monotonic loading results in the development of significant excess porewater pressures in saturated soil causing large strains under undrained conditions (e.g. Lade 1992; Chu et al., 2003; Ng, 2007; Ghiassian and Ghareh, 2008). The monotonic shearing initiating the undrained instability can be the result of drained failure (Chu et al., 2012), and the loss of shear strength associated with the undrained instability can be for only a short duration of time. However in the context of liquefaction occurring on a slope, even a short period of time with a significant drop in shear strength is sufficient to cause a large unbalanced force (i.e. downslope component of gravity minus frictional resistance), and a high initial downslope acceleration of the unstable soil mass.

Static liquefaction has been actively studied in the laboratory since the 1970's with the majority of research utilizing triaxial tests (e.g. Lade and Yamamuro, 2011; Andrade, 2009; Wanatowski and Chu, 2007). Over the years, multiple concepts have been proposed to explain and assess static liquefaction; however the most prominent is the instability line concept proposed by Lade (1992). Lade determined that sands may experience instability inside the failure surface, provided it experiences undrained conditions. It is when the soil finds itself in the stress state area between the instability line and the critical state line, called the region of potential instability, that liquefaction can occur. And yet, even though a soil lies within the region of instability it will remain stable as long as it remains drained. An increase in a pore-water pressure episode must be triggered in order to cause instability of the soil and for liquefaction to occur.

When this liquefaction behaviour does occur, triaxial tests are not able to capture the resulting behaviour of liquefaction within the context of a failure event, including the post-failure
behaviour of rapid-runout and flowslides. And yet, the catastrophic consequences of these types of events make it essential for the geotechnical community to understand the processes not only required to trigger liquefaction, but what the consequences will be when this happens on a slope.

1.3 Understanding hydraulically-induced landslides

The catastrophic consequences associated with landslides dictate that predictions regarding the triggering and subsequent behaviour of a potential failure event need to be made with as much certainty as possible. For hydraulically-induced landslides, predictions often include the use of tools based on empirical correlations made between precipitation and landslide events. These correlations create rainfall intensity-duration curves used to predict the likelihood that a landslide will be triggered for a given rainfall event within a specific area (e.g. Aleotti, 2004; Baum et al., 2012; Melchiorre and Frattini, 2012). However, these types of methods typically only provide the probability of a failure event and provide no context for understanding the triggering mechanisms nor the consequences of the failure event itself.

The complex nature of this problem has been addressed over the years with mathematical models which work to solve this problem by separating a landslide into stages, focussing on either the triggering mechanism of a landslide (e.g. Iverson, 2000; Buscarnera and Whittle, 2012;) or the post-failure and propagation stage of a landslide (e.g. Hungr, 1995; McKinnon et al, 2008) and only recently have numerical models begun to try to combine these stages (Cascini et al., 2013). This is due to the challenges posed by the complex relationship water has with the evolution of a landslide (e.g. Iverson et al., 1997; Iverson, 2000; Ghiassian and Ghareh, 2008; Leroueil et al., 2009; Olivares et al., 2009; Picarelli, 2009; Shulz et al., 2009; De Blasio, 2011) and the fact that these models require landslide events for validation. Due to the difficulty in obtaining much of the required data through back-analysis of events, or in the absence of landslide events altogether, often these models must rely on assumptions, simplifications and inferences being made. In the
absence of knowledge, such as the actual triggering mechanism, site conditions prior to failure, and precipitation history, the confidence in validity of these models significantly decreases.

This then presents a challenge. The need to further develop models that can predict the risks associated with hydraulically-induced landslides require greater understanding of the processes driving these events, and as such, actual landslide events for the validation of these models is critical. And yet the inherent nature of a flowslide is that they occur unexpectedly and are distinguished by rapid movement, making it extremely difficult to observe flowslide events in the field (e.g. Springman et al., 2009) and in the laboratory (e.g. Take, 2014).

Overcoming the challenges associated with observing landslides in the field has led to the significant use of physical modelling techniques for the observation and study of landslides, the benefits of which are their ability to control uncertainties often associated with landslides in the field (e.g. soil stratigraphy, boundary conditions, repeatability, etc.) in a controlled environment. Physical modelling techniques that are often used for observing landslides include: geotechnical centrifuges (e.g. Take et al., 2004; Ng, 2008; Bowman et al., 2010; Askarinejad et al., 2012), bench top scale flume tests (e.g. Wang and Sassa, 2007; Tohari et al., 2007; Olivares et al, 2009), laboratory-scale flume tests (e.g. Eckersley, 1990; Okura et al., 2002; and Moriwaki et al., 2004), and field-scale tests (e.g. Reid et al., 2008; Iverson et al., 2010).

Although there has been significant use of physical modelling techniques for studying landslides, similar to the challenges facing numerical models, often these techniques focus on either the triggering mechanism (e.g. erosion, hydraulic conditions, seismic activity) or the consequences of the failure event (e.g. debris runout, volume of landslide, travelling speed, etc.). Similar to the challenges in numerical modelling, the complex nature of water presents challenges for physical
modelling. For a hydraulically-induced landslide in a loose granular soil, these challenges become even more pronounced where the inherently well-drained behaviour of the material makes both triggering and observing the consequences of the landslide extremely difficult. And while separation of a landslide into individual stages for numerical and physical modelling has been shown to be common practice, observation of landslides in a holistic framework is essential for providing a fundamental understanding of the relationship between triggering mechanisms and the ensuing consequences that occur during failure.

1.4 Research Objectives

The goal of this research program was to investigate the triggering mechanisms and subsequent consequences of landslides in a loose-granular soil, using two physical modelling techniques. The specific objectives of this research are to:

- Explore the concept of base liquefaction, a novel triggering mechanism for a shear-induced failure of loose granular slopes, using high speed camera footage, image analysis, and transient porewater pressure observation.
- Study the role of antecedent hydraulic conditions on the triggering mechanism of loose granular slopes. By observing the behaviour of slope failures due to increasing antecedent conditions, the effect of rising groundwater on landslide consequences can be evaluated.
- Quantify the effect of slope inclination on the consequences of landslides triggered under the same mechanisms.
- Develop a highly-instrumented landslide testing facility for the purpose of studying the triggering mechanisms and subsequent consequences in a reduced-scale model.
- Compare the consequences of a landslide triggered in a reduced-scale model to a centrifuge test, highlighting the impact matric suction plays in a reduced-scale model on its effectiveness at modelling the triggering mechanisms and consequences of a landslide.
1.4.1 Physical Modelling Techniques

The first three research objectives were explored using geotechnical centrifuge testing, a physical modelling technique that has been extensively and successfully used by the geotechnical engineering community. Centrifuge tests allow prototype stresses and strains to be generated in a reduced-scale model by applying an enhanced gravitational field. In applying centrifugal acceleration to the reduced-scale model, scaling laws are also applied to satisfy an equivalent stress state and further details and discussion on these laws and principles can be found in the literature (e.g. Rezzoug et al., 2004; Take et al., 2004; and Garnier et al., 2007). The centrifuge tests were conducted at the C-CORE centrifuge facility in St. John's, Newfoundland.

The second physical modelling technique used in this research program was a reduced-scale model tested using a landslide flume. Flume tests have the advantage of triggering and observing failure behaviour without encountering scaling laws, and shown in Section 1.3 of this Chapter, have often been used for studying landslide behaviour. The fourth research objective is documenting the development of this flume facility for use as a reduced-scale model, a highly-instrumented landslide testing facility at the West Campus Engineering Laboratory at Queen's University. The development of the flume facility, and inaugural reduced-scale model will enable a comparison demonstration between reduced-scale models and the centrifuge tests, the fifth objective of this research program.

1.5 Organization of Thesis

This thesis is presented in manuscript format in accordance with the regulations outlined by the School of Graduate Studies at Queen's University. Chapter 1 is a general introduction, including the overarching research objectives of the research program. Chapter 2 through Chapter 5 are original manuscripts, which are followed by the overall conclusions of this research in Chapter 6.
Chapter 2 presents the results of successfully triggering static liquefaction in a loose granular material using a geotechnical centrifuge model. It was found in this paper that static liquefaction might preferentially occur in the saturated granular soil located at the base of the landslide rather than the well-draining inclined portion of the slope.

Chapter 3 builds on this by observing the triggering mechanism and resulting consequences of five landslides triggered under different antecedent rainfall conditions. The results from these centrifuge tests highlight the influence antecedent rainfall conditions have on the mobility consequences of the failure events. Chapter 4 explores the effect of slope inclination on the triggering mechanism and consequences of failure events in seven centrifuge models.

Chapter 5 combines the final two research objectives, by presenting the development of the facility and the results from the first landslide event triggered in the flume. The results from the highly-instrumented 1g flume are compared to a similar centrifuge test which showcases the advantages and disadvantages of both physical modelling techniques.

The overall conclusions of the research program are summarized in Chapter 6.
1.6 References


Chapter 2

Base Liquefaction: a mechanism for shear-induced failure of loose granular slopes

2.1 Introduction

The danger posed by landslides in loose granular materials is perhaps best illustrated in the series of catastrophic flow failures that occurred in Hong Kong in the 1970's. Prior to that time, urban developers working on the steep natural hill slopes of Hong Kong were permitted to cut benches into the slope and end-tip the spoil to create fill slopes of the loose granular completely decomposed granite soil material. Placed with little to no compaction, these fill slopes often failed under the trigger of high intensity rainfall, experiencing flow failure with severe consequences. In 1972, after an unusually heavy period of rainfall, a devastating flow failure on a loose decomposed granite fill slope tragically claimed the lives of 67 people, and over the next four years, the total landslide fatalities from similar events had reached over 175. The failure mechanism most often attributed to the triggering of high mobility landslides in these and other loose granular soil is static liquefaction (e.g. Knill et al., 1976).

Static liquefaction (liquefaction triggered from monotonic loading rather than cyclic motion such as an earthquake) is a form of deviatoric strain-softening in which an imposed monotonic loading results in the development of significant excess porewater pressures in saturated soil causing large strains under undrained conditions (e.g. Lade 1992; Chu et al., 2003; Ng, 2007; Ghiassian and Ghareh, 2008). The monotonic shearing initiating the undrained instability can be the result of drained failure (Chu et al., 2012), and the loss of shear strength associated with the undrained instability can be for only a short duration of time. However in the context of liquefaction occurring on a slope, even a short period of time with a significant drop in shear strength is
sufficient to cause a large unbalanced force (i.e. downslope component of gravity minus frictional resistance), and a high initial downslope acceleration of the unstable soil mass.

In order for the critical state strain-softening associated with static liquefaction to occur, the soil must be contractile (i.e. have a sufficiently high void ratio with respect to its confining stress, defined by the state parameter of Been and Jefferies, 1985), be subjected to a monotonic loading trigger, and be sufficiently saturated to permit the generation of excess pore water pressures upon loading. However, for granular soil on a slope it is this third condition that is perhaps the most difficult to achieve. Slopes consisting of granular material are inherently well drained, as they consist of a high permeability material, placed with an inclination that encourages high hydraulic gradients, which act to drain the slope. As a result, reaching a zone of saturated granular flow of significant thickness on a granular slope would require very high groundwater seepage flow rates that may or may not occur under typical groundwater and rainfall infiltration conditions. It is hypothesised in this paper that a more likely location for static liquefaction to potentially occur would be in the granular soil located at the base of the landslide which would be easier to saturate. The change in slope geometry at this location leads to hydraulic conditions that favour saturation at a lower flow rate than would be required to trigger liquefaction failure in an infinite slope of the same inclination. This concept is illustrated graphically in Figure 2-1. Figure 2-1a shows an infinite slope of granular soil on an impermeable bedrock foundation, which, despite being subjected to a total groundwater flux (Q), results in a small thickness of saturated granular material at risk of liquefaction. However, slopes in practice are not infinitely long. Therefore, if the geometry of the slope were to be modified by adding a toe to the slope (Figure 2-1b), the resulting change in hydraulic gradient in the toe region could result in a thick zone of saturated granular soil generated at the same total flux of groundwater (Q) that was applied in Figure 2-1a. If this soil were sufficiently loose, and if a rainfall or seepage induced local toe failure were to
provide a monotonic shearing trigger, it is possible that the base of the landslide may be at higher risk of experiencing static liquefaction than the more well drained inclined portion of the slope. If this region of the toe were to liquefy, it is further hypothesised that this loss of support to the slope would create a landslide as shown by the dotted line in Figure 2-1b.

The objective of this paper is to test experimentally the hypothesis that base liquefaction might be an admissible mechanism for static liquefaction in loose granular slopes. This will be achieved by reproducing the scenario used to describe the hypothesis in Figure 2-1b in a reduced-scale physical model centrifuge experiment to attempt to trigger a static liquefaction event and using measurements of pore pressure and volumetric strain to assess whether liquefaction has occurred.

2.2 Centrifuge Model

2.2.1 Geometry and slope material

Physical modelling has been successfully used to investigate a wide range of different failure processes in granular slopes (e.g. Eckersley 1990; Take et al., 2004; Ng, 2008; Lee et al., 2008; Olivares and Damiano, 2007; Askarinejad et al., 2012). In this study, the slope geometry of the physical model was chosen to test the hypothesis of base liquefaction as described in Figure 2-1b. As a result, the properties of the centrifuge model chosen for the study (Figure 2-2) were designed to provide conditions to make the soil layer as susceptible to liquefaction as possible. Firstly, in order for liquefaction of the base of the slope to occur, the slope geometry must include a layer of contractile soil below the toe at the base of the slope. As a result, a 50 mm thick layer of loose soil was provided for both the slope and the base of the model. This depth of the soil in the model was chosen to represent a typical shallow slope when tested at a test acceleration of 30 gravities (i.e. 1.5 m soil layer at equivalent prototype scale). Above the base layer, the slope was inclined at an angle of 30° with a slope length of 250 mm and width of 300 mm. A slope angle of 30° was chosen for the centrifuge model as it falls within the range of 13 - 36° angles that
Jefferies and Been (2007) reported to be susceptible to liquefaction. The foundation for the model slope was an aluminium plate which was designed to represent a frictional bedrock base layer and act as an impermeable boundary. The face of aluminium was coated with a roughness (waterproof sandpaper) to increase the interface friction angle between the aluminium and sand. A moldable sealing compound (Duxseal) was applied to the sides of the bedrock/soil interface to ensure a no flow boundary condition between the model bedrock and the transparent side walls of the testing chamber housing the experiment.

A fine uniform sand (U.S. Silica F110 Ottawa Sand, Figure 2-3) was selected to be the granular slope material in order to further encourage the liquefaction susceptibility of the model slope. The properties of this material, including its critical state parameters at low confining stresses, have been well characterized by Santamarina and Cho (2001) and are reproduced in this paper as Table 2-1. To accompany this particle size and shear behaviour data, additional hydraulic conductivity tests were performed in the laboratory using constant head boundary conditions with the resulting saturated hydraulic conductivity found to be $1 \times 10^{-4}$ m/s.

**Table 2-1. Properties of Ottawa F-110 Sand**

<table>
<thead>
<tr>
<th>Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$e_{\text{max}}$</td>
<td>0.848</td>
</tr>
<tr>
<td>$e_{\text{min}}$</td>
<td>0.535</td>
</tr>
<tr>
<td>$D_{50}$ (mm)</td>
<td>0.12</td>
</tr>
<tr>
<td>$D_{10}$ (mm)</td>
<td>0.081</td>
</tr>
<tr>
<td>$C_u \left( D_{60} / D_{10} \right)$</td>
<td>1.62</td>
</tr>
<tr>
<td>$C_c \left( D_{30} / D_{10} \cdot D_{60} \right)$</td>
<td>0.99</td>
</tr>
<tr>
<td>$G_s$</td>
<td>2.65</td>
</tr>
<tr>
<td>Friction Angle, $\phi_{cs}$</td>
<td>$31^\circ$</td>
</tr>
<tr>
<td>Intercept of CSL at 1 kPa in $e$-$\log p'$</td>
<td>0.937</td>
</tr>
<tr>
<td>Slope of CSL in $e$-$\log p'$</td>
<td>0.077</td>
</tr>
</tbody>
</table>

$^1$Santamarina and Cho (2001)
Following the technique of Davoodi (2008), sand was carefully placed into the model container to achieve a very loose initial state. This was achieved by first mixing the sand to a gravimetric moisture content of 7% to encourage the development of matric suction in the fine sand (of approximately 5 kPa). The wet sand was then rained through a No. 5 sieve from an elevation range of 100 mm to 150 mm above the model bedrock surface. The sand was placed in ten 5 mm lifts, and to maintain the loose soil structure there was no tamping of the material between lifts or smoothing of the surface at the end of sand placement. These efforts resulted in a sand layer with a very loose initial state (void ratio, $e = 1.72$). This initial void ratio is significantly greater than maximum void ratio ($e_{\text{max}}$) for this sand of 0.848 due to the metastable “sandcastle” structure supported by the matric suction in the fine sand. The reasoning for such a high initial void ratio is to create a slope in its loosest possible state (i.e. the highest possible void ratio) to encourage liquefaction to occur. Whereas the centrifuge model was assembled on the laboratory floor, the testing of the slope was conducted at the elevated acceleration of 30 gravities. As a result, the void ratio was chosen to be sufficiently high that the reduction in void ratio reduction during gravity turn-on would leave the sand layer in its loosest possible state under the higher imposed body stresses. The void ratio reduction that was observed in the physical model during gravity turn-on resulted in a final pre-landslide void ratio of 1.06, corresponding to a dry density of 964 kg/m$^3$, still significantly above the critical state line as defined by Santamarina and Cho (2001).

2.2.2 Hydraulic boundary conditions
In order for liquefaction of the base of the slope to occur, the geometry of the slope must include a reduction in the hydraulic gradient of groundwater flow at the toe of the slope to permit ponding in the layer of contractile soil below the toe at the base of the slope. This was achieved in the physical model by extending the base layer of loose granular soil by approximately 250 mm past the toe of the slope to control the hydraulic gradient in this region (Figure 2-2).
An applied flux boundary condition was provided at the top of the slope to provide groundwater seepage flow. This boundary condition was implemented in the model using a metering pump (i.e. a flow pump that can be controlled by an analog voltage input signal) discharging into a buried weir embedded within the slope. The buried weir was cut to the curvature of the curved acceleration field across the plane-strain width of the model chamber and was used to increase uniformity of groundwater seepage in the model. The flow rate, \( Q_{\text{IN}} \), was varied by controlling the analog voltage input to the control channel of the pump. It was recognized that the original calibration (at 1g) of the pump may be inaccurate under the elevated self-weight of the pump components and the higher back pressure at the outflow port due to the self-weight of the water column leading to the buried weir. Therefore the pump was calibrated at 30 g prior to running the test to ensure an accurate relationship between the applied pump voltage to seepage rate during the test.

A zero pressure head boundary condition was promoted at the end of the base, in the form of a gravity drain (Figure 2-2), allowing water to flow back to the settling pond (\( Q_{\text{OUT}} \)). The water was re-circulated from the settling pond back to the top of the slope to reduce spillage in the centrifuge pit. The settling tank allowed any soil particles suspended in the out flowing water to settle before the final stage of filtration at the pump intake.

Take and Bolton (2004) describe the conflicting scaling laws associated with inertial events and diffusion processes in centrifuge modelling. In this study, water was used as the pore fluid, as a greater emphasis was placed on modelling the seepage processes leading to landslide triggering than quantifying the mobility and runout of the resulting landslide. As a result, the shear-induced
porewater pressures will dissipate faster than would be the case at field-scale, resulting in a quicker return to stability in the physical model.

### 2.2.3 Instrumentation

The instrumentation design for the experiment must be capable of determining whether the slope has liquefied or not. In this regard, there are two distinct characteristics of static liquefaction slope failures that need to be observed and quantified, should such a failure occur. The first is that instrumentation must be in place to measure shear induced porewater pressures. The second indicator of liquefaction is related to the observed deformations in the slope, the volumetric strain behaviour of the landslide mass and the resulting velocity of any instability event. Two specific instrumentation and analysis methods were used to capture these two liquefaction criteria and are presented below.

Eight GE-Druck PDCR-81 pore pressure transducers (PPTs) were installed in the physical model to capture the porewater pressures (PWPs) applied and developed during the test,. The locations of the eight sensors were chosen, based on two considerations. The first factor was the embedment depth of the sensors within the sand. Placing the PPTs within the sand would allow for pore water pressures to be monitored at varying soil depths. Placing the sensors within the sand would result in a restriction to movement during shearing due to the size of the sensors in relation to the overall size of the model and total sand depth. Alternatively the sensors could be placed flush with the bedrock/sand interface, with the body of the sensor resting in the foundation layer, and this mode was selected for the test. This provided PWP readings at the bedrock/sand interface and caused no restriction to sand movement. The spacing of the eight sensors along the bedrock/sand interface was also selected strategically. PPT1 was placed such that it would measure the water pressures at mid-height of the slope, whereas PPT2 through PPT8 were located
to capture the seepage response near and along the base of the slope (see Figure 2-2). This would provide more detail in the area of interest and the location where the susceptibility to liquefaction was hypothesised to be the greatest.

The second factor needed to quantify whether or not liquefaction occurred was the slope movement during failure. Having only pre and post failure images of the landslide mass makes it very difficult to define liquefaction during failure. Therefore, it is necessary to capture movements during failure to quantify whether liquefaction was triggered during the test or not. The method selected to study slope movement was to apply high speed imaging (1000 frames per second, fps) analyzed using the Particle Image Velocimetry (PIV) code developed by White et al. (2003). Slope movement and shear and volumetric strains can be calculated using images taken during failure and analyzed to determine if liquefaction occurred.

Figure 2-4 is the first image taken from the image series captured during slope failure of the physical model. The photographs were taken from outside the plane-strain centrifuge model box using a Phantom V9.0 high-speed camera, recording images at a frame rate of 1000 fps. The size of each captured image is 1632 x 1200 pixels, which corresponds to a field of view (FOV) of 345 mm x 254 mm. The location of the FOV (Figure 2-2) was selected to capture a portion of both the slope and the base of the model centered on the area of greatest interest. The light source for the images was provided by two wide angle DC voltage (i.e. flicker free at 1000 fps) light sources providing a uniformly lit surface across the camera FOV (Kaewsong, 2009). The camera, being outside the plane-strain box, meant that the image was taken through a 76 mm thick Plexiglas window. An additional glass panel (6.5 mm in thickness, reducing model width to 287 mm) was also placed inside the model on both sides of the plane strain width to reduce the interface friction angle between the soil and the transparent side walls to approximately 13° (Saiyer, 2011).
Because the camera lens experiences self-weight deformation under elevated gravity, the images capture the box experiencing movement when centrifugal spinning begins. As described by White et al. (2003), this can be accounted for (while simultaneously eliminating camera lens distortion and refraction issues through the thick window, etc.) by tracking the apparent movements of a series of non-moving targets placed on the inside of the transparent window. These control makers are shown in Figure 2-4 as the numerous white circles with black centers. The bottom material layer seen in Figure 2-4 is the impermeable "bedrock" base.

In its natural state, the F110 Ottawa sand used in this test has a uniform white colour. When using PIV to track the sand movement, a uniform coloured material can lead to erroneous measurements of deformation (e.g. Dutton et al., 2011). The contrast of the Ottawa F110 sand at the transparent window was increased by sprinkling Ottawa sand that had been dyed black after each lift layer to create a high contrast pattern to overcome this limitation of the chosen sand (seen in Figure 2-4). A mesh of PIV patches (e.g. S1 and B1 patches in Figure 2-4) were defined in the image coordinates and the location of each individual patch was measured every 0.001 s during the failure process using GeoPIV analysis tools (White et al., 2003).

2.3 Testing Results

2.3.1 Model response to applied groundwater flow
Once the construction of the soil model was completed, the model slope was loaded onto the centrifuge and was subjected to a stepwise increase in centrifugal acceleration and the first seepage flux was introduced upon reaching 30 g. A graphical representation of the stepwise increase in total flux imposed on the slope during the test is presented in Figure 2-5a. The total flow is quantified in model scale with units of mL/min, and is plotted versus elapsed time in seconds. As seen in Figure 2-5a, the stepwise increments of total flux were reduced as the slope
was taken closer to failure. The PWP recorded by the PPTs from before spin-up until post failure are plotted in Figure 2-5b. At the start of the test, and prior to gravity turn on (t=0-100 s), all the sensors show readings of approximately 0 kPa. The moisture in the soil on the slope began to drain out of the slope towards the base (time waypoint “A”) as the centrifuge acceleration increased to 30 g. This resulted in small increases in porewater pressures (up to 5kPa) along the base PPTs prior to the application of groundwater seepage flux. It should be noted that the results from PPT7 indicate that this sensor malfunctioned prior to testing and therefore is excluded from the results.

Both PPT1 and PPT2 show a smaller increase in porewater pressure for the first applied flux step of 75mL/min than PPT3-PPT8. This is due to the difference in sensor location. PPT1 and PPT2 are both found on the slope of the model where the soil is experiencing a greater influence of drainage due to gravity, compared to PPT3-PPT8. This trend of lower PWP values at PPT1 and PPT2 continues for all 4 applied flux steps. The equalization of PWP readings is cut short after the fourth and final increase in flux (reaching 160 mL/min) at a time of 2040 seconds. After only twenty seconds at the final flux rate of 160 mL/min a failure event occurs. Excess porewater pressures generated from shearing can be observed as a PWP spike at a time of 2060 seconds in Figure 2-5b. This failure will be discussed in detail in an upcoming section of the paper.

Each of the applied flux stages has been labelled (A through D) in Figure 2-5b at a point when the PPTs are approaching their equilibrium steady state seepage values. The values of total head measured at these time steps are plotted as discrete solid markers on a side profile view of the physical model (Figure 2-6). An analysis of the steady-state seepage conditions imposed on the model was performed in Seep/W and are included in Figure 2-6 as lines of total head. This
comparison indicates that the measured total heads are internally consistent with the hydraulic boundary conditions (applied flux stages) imposed in the physical model.

There is no applied steady state seepage to the model at time A (t=500s). However, there is a measurable amount of groundwater that has collected along the base from the draining slope during spin-up. This is reflected in the total head values of the base PPTs.

The first application of steady state seepage has been applied at time step B. There is a noticeable increase in the total head values from those in step A. The rising total head values along the base during step B to C is from the low hydraulic gradient along the base (1° slope) enabling water to back up. The rise is also seen in PPT3, and to a lesser extent in PPT2, which are on the slope but near the toe. However PPT1 is located high enough up the slope that it is yet to be affected by the increasing steady state seepage rate. With the steady state seepage increases between stage C to D, the seepage conditions are now prime for liquefaction with a fully saturated zone along the base. This is a very important observation with respect to the hypothesis of this paper: the target seepage rate applied has thoroughly saturated the base of the physical model while the slope remains well-drained.

The transient PWPs at the time of failure (the PWP spike in Figure 2-5b) are shown in greater detail in Figure 2-7. Of note is the length of time shown on the x-axis in Figure 2-7. The time window shown is only 1 second, yet it is still able to capture in entirety the excess generated PWPs (plotted on the y-axis). PPT3 and 4 show both the highest magnitude and the earliest PWP peaks. The spikes in Figure 2-7 are the result of shear-induced PWPs and not from a further increase in flux to the system. This indicates that the soil has experienced shearing movement, and has generated these excess porewater pressures as a result. The time steps in Figure 2-7 (E-I)
are the corresponding times used for plotting the total head at each PPT versus time in Figure 2-8. Profiles of transient shear-induced total head during landsliding. Similar to Figure 2-6, the total heads at the different time steps are plotted at the respective PPT locations. The first time step E in Figure 2-7 shows the PWP readings from the final flux application of 160 mL/min immediately prior to failure of the slope. Despite the short period of time of the excess generated PWPs (< 1s) from time step E - H, the total heads at PPT3-8 all had time to exceed the height of the soil surface. The magnitude of these pore water pressures is perhaps best communicated through the use of the porewater pressure ratio \( r_u \), defined in Equation 2-1 as the ratio of the pore water pressure to the total stress in the soil at that location,

\[
    r_u = \frac{u}{\gamma h}
\]

where \( u \) is the porewater pressure (kPa), \( \gamma \) is the unit weight (kN/m\(^3\)) and \( h \) is the depth of the soil (m).

The consequence of a \( r_u \) ratio approaching unity would be to have little or no effective confining stress acting on the soil. If such conditions were to occur, the soil’s shear resistance can temporarily drop significantly below the static shear stress acting on the slope, leading to large accelerations under unbalanced downslope forces. The porewater pressure ratio was calculated at the location of each PPT for time steps A through H (from Figures 5b and 7) and is presented in Table 2-2.

The \( r_u \) values calculated in the base soil layer below the toe of the landslide indicate that this soil has liquefied (\( r_u=1 \)). In contrast, the \( r_u \) values calculated on the slope during the failure event did not experience sufficient shear-induced PWPs to trigger liquefaction (\( r_u=0.22 \)). Thus the loose sand on the slope has a sufficiently small volume of saturated soil to be able to rapidly dissipate the excess generated PWPs despite the slope experiencing failure. In contrast, the loose granular
soil at the base is almost entirely experiencing $r_u$ values that would imply liquefaction (bolded values in Table 2-2 are $r_u > 0.75$). The pore pressure data therefore indicate that in slope geometries such as that tested in the physical model, shear induced pore pressures sufficient to trigger full or partial liquefaction may be more likely to occur at the base of the landslide toe than on the well-drained slope.

**Table 2-2. List of $r_u$ values\(^1\) at multiple time steps**

<table>
<thead>
<tr>
<th></th>
<th>Applied Flux</th>
<th>PWP Spike</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td><strong>PPT’s Slope</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.01</td>
<td>0.10</td>
</tr>
<tr>
<td>2</td>
<td>0.01</td>
<td>0.29</td>
</tr>
<tr>
<td>3</td>
<td>0.19</td>
<td>0.58</td>
</tr>
<tr>
<td><strong>PPT’s Base</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0.21</td>
<td>0.52</td>
</tr>
<tr>
<td>5</td>
<td>0.16</td>
<td>0.49</td>
</tr>
<tr>
<td>6</td>
<td>0.23</td>
<td>0.52</td>
</tr>
<tr>
<td>7</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>0.14</td>
<td>0.43</td>
</tr>
</tbody>
</table>

\(^1\)Bold values indicate $r_u > 0.80$

### 2.3.2 Evolution of failure

The experimental design of the centrifuge test ensured that the landslide failure process can be captured in slow motion using high-speed imaging, enabling further quantitative measurements of displacements, velocities, and strains using GeoPIV. Vectors of total displacement describing the evolution of the slope movement at different periods of time throughout failure are presented in Figure 2-9. This figure provides a description of the state of slope failure at six time increments covering the time of initial movement until the time where landslide movement has come to a complete stop. The total time of the failure event captured in images was 0.167 seconds, with the
The physical model showing signs of movement for 0.118 seconds (3.54 seconds at prototype scale). Also shown in the inset of Figure 2-9, as is the outline of the physical model, where the camera's field of view (FOV) is identified as the area shown for the sub-figures. The FOV was divided into 308 PIV patches (or measurement regions sometimes called subsets) for the GeoPIV analysis. The movement of each of these locations was tracked and the total displacement of each location calculated. The total displacement vectors plotted in Figure 2-9 are the displacement vectors of each patch up to and including the period of time indicated in the figure. A scale vector of 25mm is shown in Figure 2-9b.

The displacement vectors in Figure 2-9a are at a time of 0.002 seconds. This time step indicates that no movement has yet begun. It was included in the time sequence to show the initial locations of the subsets being tracked. By the time the second time step is plotted at a time of 0.025 seconds (Figure 2-9b), a small local failure has begun to emerge at the toe. What is clear from this initial movement shown in Figure 2-9b, is that the location of initial failure is on the slope of the model, and not in the soil at the base of the slope. The third time step (Figure 2-9c) goes on to show the evolving failure which progresses from a small local failure to one with a deeper slip surface and mobilizing a greater volume of soil. The failure surface continues to progress and by step four (Figure 2-9d) the loose granular soil at the base has begun to move. The landslide continues to shear along this deeper failure surface through the final two time steps (Figure 2-9e,f) to the point where the sliding soil passes outside the camera’s FOV.

The geometry and boundary conditions of the test programme were designed to encourage slope failure and to provide favourable conditions for static liquefaction to occur, if applicable. Therefore it should come as no surprise that the slope failed. However the evolution of failure captured in the physical model yields a particularly interesting story of how progressive failure
can evolve in granular slopes. From a simple analysis of the failure event over time, it is seen that what began as a small failure on the sloping portion of the model turns into a much larger failure event, complete with a deeper slip surface. The behaviour of the slope and the base are investigated in more detail by selecting two regions, one each on the base and the slope (Patches B1 and S1 on Figure 2-4, respectively), to compare and contrast their behaviour through time.

The displacement-time and velocity-time relationships for patches B1 and S1 are shown in Figure 2-10. The total model-scale displacement of the slope during the landslide was slightly more than 60 mm, while the base moved slightly more than 40 mm. More importantly, the displacement results in Figure 2-10a show that the toe region of the slope (S1) moves significantly in both the x and y directions before the base soil (B1). This data presents quantitatively the timing of the initial local failure at the toe, which occurred before any deformation was observed in the base soil. The base begins to move only after the slope patch has been moving 0.02 second and having already accumulated 12 mm of horizontal displacement. The failure event as observed in Figure 2-9 appears to begin as a small rotational failure. Additionally, it is seen in Figure 2-10a that the x-displacements continue to increase as the slope failure progresses well after the y-displacements have stopped. The slope therefore continued to travel after it had reached the base of the model. Had this failure remained a small localised rotational failure, the soil comprising the failing slope would have come to a rest on top of the base foundation soil. Instead, the progressive nature of the failure into this base zone is consistent with this monotonic shearing of the loose saturated sand at the base of the slope, resulting as a significant loss of shear strength of the base material, and thereby temporarily removing support to the remainder of the slope.

The velocity of the two selected regions of the slope and base are shown in Figure 2-10b. Similar to the magnitude of the displacement results in Figure 2-10a, the slope patch reaches a velocity of
What is interesting is that despite the time difference when the two patches begin moving, both patches reach their maximum velocity within 0.05 seconds of each other (at 0.065 s), meaning the base has accelerated at a faster rate than the slope. Once the slope patch reaches its peak velocity it almost immediately begins to slow down. In contrast, the base patch maintains its maximum velocity with minimal change of rate for a length of time equal to 0.04 s. At this point of time (0.095 s), both the slope and the base now have a similar velocity and thereafter decrease in velocity at the same rate, coming to a stop at the same time.

2.3.3 Shear strain and volumetric strain analyses

The pore pressure data indicate that the loose soil on the inclined portion of the slope experienced shearing with contractile volumetric tendencies, but that an undrained response to this attempted volume change was mitigated by insufficient saturation. As a result, minimal shear-induced pore pressures were generated. In contrast, the soil at the base of the slope experienced shearing later in the failure process but with significantly large shear-induced pore water pressures to be indicative of possible liquefaction. If this soil were to have experienced liquefaction in truly undrained shearing, these shear strains should be accompanied with negligible volumetric strain. In this manner, the nature of the observed coupled shear and volumetric behaviour of the soil can shed additional light on the mechanics of failure on the slope and the base of the landslide.

The shear and volumetric behaviour of the soil at various locations within the landslide mass can be calculated using image analysis. The locations (subsets) of displacement measurements in the digital images captured by the high speed camera can be defined at any selected point in the image. To calculate strains, the subsets can be arranged to act as nodes at the corners of strain calculation elements. Two triangular strain elements were defined: one about a centroid containing the loose granular soil on the inclined portion of the slope and another about a centroid
containing loose granular soil within the base portion of the slope. Further refinement of the calculated strain data used multiple triangle elements rotated about each centroid to minimize the noise within the strain data.

The calculated shear and volumetric strains within the inclined portion of the slope and the base region of the slope are presented in Figure 2-11. The shear strain of the slope and the base patches are plotted versus time in Figure 2-11. The slope element begins experiencing shear strain before the base does (expected from the displacement and velocity results). The base experienced similarly small shear strains (1.0 – 1.5 %). This does not indicate that liquefaction did not occur. Rather, as described by Lade and Yamamuro (2011), the initiation of instability leading to liquefaction is from small shear strains.

Compared to the similar shear strain behaviour, the volumetric strain differs between the two patches. The volumetric strain of the slope patch is plotted versus time in Figure 2-11b and shows a decrease in volume during the failure event. This indicates that the slope is experiencing both contraction and partially drained conditions during shearing. This matches the $r_u$ values seen in Table 2-2, where the slope did not generate large excess pore water pressures due to having the ability to reduce volume during shearing. In contrast, the base patch shows shearing at zero volumetric strain, indicating an undrained response – a response that while not essential for liquefaction (Wanatowski et al., 2010), implies liquefaction.

2.4 Applicability of Back-Analyses to Liquefaction Events
Numerical analyses of the failure and post-failure propagation behaviour of landslides which experience a significant reduction of mean effective stresses and as a result transition from slide to flow is a highly challenging numerical modelling task. Conventional approaches such as the limit equilibrium method (LEM) have been shown to only be applicable for the analysis of the
initial stage of failure as they consider localised shear surfaces rather than the more diffuse failure of liquefied soil and do not consider the propagation stage of landslide runout (Cascini et al., 2010, 2013). Recent advances on the analysis of both the failure and post-failure stages of landslides within a unified framework (i.e. both triggering and propagation) have been made by Cascini et al. (2013) using a hydromechanical coupled finite element model. However, a conventional limit equilibrium approach has been adopted in this paper to make the point that even a simple analysis of only the initial triggering event of this type of landslide can be potentially misleading.

The typical strategy used in the conventional limit equilibrium back analysis approach is to set the factor of safety set to unity so that the mobilized strength at failure of the slope can be back-calculated based on a) the measured location of the failure surface as observed in a detailed site investigation, and b) credible estimates of the pore pressure conditions applicable at the time of landslide triggering.

Such a back analysis was performed using Slope/W (GeoStudio, 2007) to explore the applicability of such an analysis to the complex progressive liquefaction failure mechanism observed in the physical model. This limit equilibrium analysis was performed using the Morgenstern-Price method, the assumption of saturated soil behaviour, the measured final slip surface and with full knowledge of the values of pore water pressure measured milliseconds prior to landslide triggering (Figure 2-12a). However, despite this high degree of certainty in the back analysis, the back-calculated friction angle for FOS=1 was calculated to be 22°. This value is clearly not plausible for a sand (Table 2-1 indicates that the friction angle of the sand is 31°), and would lead an analyst to wonder where the friction of the sand had disappeared. The wording of “missing friction” in this previous sentence is a misnomer. Instead, this erroneous result is due to
the inappropriate use of the static pre-shearing pore water pressures with the post-liquefaction failure surface. With the benefit of hindsight provided by the slow motion footage of the progressive failure of the slope, it is known that the initial triggering event was a shallow slip surface local to the toe of the slope. If instead a back-analyses in Slope/W is performed using this failure surface (Figure 2-12b), a more credible friction angle at failure of 30° is calculated. This back-analyses now conforms with the known value of 31° reported by Santamarina and Cho (2001) and included in Table 2-1.

Unfortunately, liquefaction failures in loose granular soils that occur in the field are not accompanied with slow motion footage of the behaviour of a plane-strain section of the landslide. In these cases, the initial failure event will be unknown. However, the back analysis case history provided here indicates that the use of the static pre-shearing pore water pressures (i.e. seepage and/or infiltration analyses) with the measured position of the failure surface will be likely to lead to erroneous back-calculations of mobilized strength due to the progressive nature of the liquefaction failure event. A high degree of caution should therefore be exercised in the interpretation of back-analyses of these highly complex failures using conventional limit equilibrium analyses.

2.5 Conclusions
In order for the deviatoric strain-softening associated with static liquefaction to occur in a landslide, the soil must be contractile (i.e. have a sufficiently high void ratio with respect to its confining stress), be subjected to a monotonic loading trigger, and be sufficiently saturated to permit the generation of excess pore water pressures upon loading. It was hypothesised that a more likely location for static liquefaction to occur would potentially be in the more easily
saturated granular soil located at the base of the landslide rather than the well-drained inclined portion of the slope.

This hypothesis was tested using the technique of geotechnical centrifuge modelling in which a loose granular slope was brought to failure under a step-wise increase in groundwater flux. The elevated pore water pressures eventually led to a small localised failure at the toe of the slope. This toe failure acted as the monotonic loading trigger to shear the loose contractile saturated sand at the base of the slope and cause liquefaction to occur in the base region. The subsequent movement of the base soil then caused a loss of support to the non-liquefied soil on the inclined portion of the slope as the landslide progressed.

Historically, liquefaction failures have been very difficult to replicate experimentally in physical model tests. However, in the present study both the dynamic pore pressure ratios during failure ($r_u=1$) and the lack of volumetric strain during shearing of the base soil (i.e. shearing with no volume change) observed in geoPIV analyses indicate that liquefaction occurred in the model. The difference between this test and many other studies was the presence of a loose granular base layer which reduced the hydraulic gradient (i.e. making saturated conditions more likely) providing a liquefaction prone soil region at the base of the slope. Thus the likely location for static liquefaction to occur for this particular geometry, would be in the more easily saturated granular soil located at the base of the landslide, rather than the well-drained inclined portion of the slope. This adds weight to the argument that this region of the slope must therefore also be considered for densification or the provision of drainage measures, when considering landslide remediation options.
Finally, a limit-equilibrium back analysis was performed of the observed failure. This analysis indicates that these types of analyses should be used with great caution post liquefaction events as the progressive nature of the event following triggering will be likely to lead to erroneous back-calculations of mobilized strength. This is due to the incompatibility between the static preshearing pore water pressures (i.e. seepage and/or infiltration analyses) with the measured position of the failure surface that was achieved based on the excess shear-induced pore water pressures during liquefaction.

2.6 References


a) Infinite Slope

b) Slope with Base

Figure 2-1 Effect of base of slope on hydraulic conditions leading to the zone of loose saturated granular material hypothesized to be prone to liquefaction.
Figure 2-2 Physical model of landslide illustrating slope geometry, instrumentation layout and hydraulic boundary conditions.
Figure 2-3 Particle size distribution of F-110 Ottawa Sand.

Figure 2-4 Field of view from high-speed camera illustrating two regions (patches) tracked in the PIV analysis of slope displacements [origin of axes is located at the bottom left of model container as shown in Figure 2-2].
Figure 2-5 a) Applied flux boundary condition applied to slope model, and b) resulting pore water pressures response.
Figure 2-6 Profiles of total head during steps of steady state seepage observed in physical model and numerical model.
Figure 2-7 Generation of transient shear-induced porewater pressures during landsliding.
Figure 2-8 Profiles of transient shear-induced total head during landsliding.
Figure 2-9 Vectors of total landslide displacement describing evolution of landslide, starting from a small localised failure at the toe and progressing into a larger, more high velocity liquefaction event.
Figure 2-10 Displacement and velocity time histories for soil regions on the base and the sloping portions of the landslide
Figure 2-11 Shear and volumetric strain time histories for soil regions on the base and the sloping portions of the landslide.
Figure 2-12 Challenges associated with back-analyses of static liquefaction events describing a) erroneous back-calculated mobilised strength when the final slip surface is used, and b) the correct back-analysis using the small initial localised failure surface.
Chapter 3

Effect of antecedent groundwater conditions on the triggering of static liquefaction landslides

3.1 Introduction

Real-time early warning systems for shallow landslides (failure depth less than 2 m) are typically built upon real-time measurements and forecasts of rainfall, and empirical correlations between past patterns of rainfall and landslide occurrence. The most commonly applied correlation is between rainfall intensity, rainfall duration, and the occurrence of landsliding. These correlations are often derived by combining a database of the timing of past landslide events with a continuous record of local rainfall to identify threshold combinations of rainfall intensity and duration (e.g. Caine, 1980; Guzzetti et al, 2008; Giannecchini et al., 2012) which cause landslides to be triggered (Figure 3-1a). An alternative strategy is to build the early warning system upon empirical correlations between rainfall, antecedent rainfall (a proxy for pre-existing suctions or groundwater levels in the slope), and landslide occurrence (Figure 3-1b). As demonstrated by Terlien (1998), the choice of time period over which to define the antecedent rainfall (i.e. number of days the cumulative rainfall is summed) is critical to the success of this method in identifying threshold combinations leading to landslide triggering.

The theoretical basis for these empirical threshold relationships has been thoroughly investigated using analytical and numerical unsaturated infiltration models in the literature. The stability of unsaturated soil slopes subject to transient infiltration and seepage has been investigated in detail through the use of numerical modelling by Ng and Shi (1998). Their findings illustrate that rainfall infiltration causes a transient reduction in matric suction, an increase in moisture content, and an increase in permeability in unsaturated zones. At sufficiently high rainfall intensities, a
perched water table can be formed above the main water table, potentially reducing stability to the point where landslides may be triggered in certain circumstances. The parametric analyses performed by Ng and Shi (1998) illustrate that the location of the initial ground water table also has a significant effect on the stability of a slope. In soil deposits with a very deep main water table (or free drainage at the base), numerical models (e.g. Ng and Shi, 1998; Terlien, 1998), laboratory infiltration column tests (e.g. Siemens et al., 2013), and field measurements (e.g. Matsushi and Matsukura, 2007) indicate that infiltration at low rainfall rates can occur for long durations with the soil slope remaining in suction – conditions that may permit the slope to remain stable despite the transient reduction in suction associated with rainfall. However, in conditions where groundwater is permitted to pond and collect at a soil-bedrock contact, these low infiltration rate and long duration conditions may result in the development of positive pore water pressures (e.g. Matsushi and Matsukura, 2007), and landslide triggering. The permeability contrast at the soil-bedrock contact and the depth to the water table therefore have considerable influence on the combinations of rainfall events which lead to failure.

Whereas the relationships shown in Figure 3-1 describe whether certain combinations of rainfall and pre-existing groundwater levels are of elevated risk of landslide triggering, not all combinations leading to landslide events necessarily have the same consequences in terms of landslide mobility (velocity and distal reach of the landslide). In this paper it is hypothesised that the mobility of landslide events triggered at high antecedent groundwater conditions is potentially higher than combinations with drier initial conditions. This hypothesis is based on recent physical model experiments which have been used by Take and Beddoe (2014) to investigate whether static liquefaction might preferentially occur within the easily saturated granular soil located at the toe of a landslide rather than the well-drained inclined portion of the slope.
Static liquefaction is a form of deviatoric strain-softening in which an imposed monotonic loading results in the development of significant excess porewater pressures in the saturated soil causing large shear strains to develop under undrained conditions (e.g. Lade 1992; Robertson et al., 2000; Chu et al., 2003; Ng, 2007; Ghiassian and Ghareh, 2008; Baki et al., 2012). In order for the deviatoric strain-softening associated with static liquefaction to occur, the soil must be contractile (i.e. loose), be subjected to a monotonic loading trigger (e.g. a localised rainfall-induced failure), and be sufficiently saturated to permit the generation of excess pore water pressures upon shearing. Out of all of the combinations of rainfall conditions leading to failure shown in Figure 3-1b it is hypothesised that the most susceptible to the high velocity and long distal reach associated with liquefaction events are those triggered when the soil is at high antecedent groundwater values as these conditions satisfy the need for a high degree of saturation within the soil.

The objective of this paper is to quantitatively test this hypothesis using the technique of geotechnical centrifuge modelling in which five identical slope models will be subjected to five differing antecedent groundwater conditions to observe the relationship between antecedent conditions and the velocity and travel distance of the resulting landslides.

3.2 Materials and Methods

3.2.1 Experimental Program
Physical models have successfully been used to investigate a wide range of slope failures and landslides in order to better understand their behaviour (e.g. Eckersley, 1990; Take et al., 2004; Ng, 2007; Lee et al, 2008; Olivares et al., 2008; Askarinejad et al., 2012; Take and Beddoe, 2014; Take, 2014). In this paper, physical modeling will be used to determine how antecedent groundwater levels prior to the failure rainfall event affect the landslide mobility with respect to
displacement, velocity and total volume of the landslide. To achieve this objective five physical model tests were performed under a range of antecedent groundwater conditions. The first experiment will investigate the scenario where the slope has no antecedent groundwater flow (Figure 3-2a) and only rainfall from the current storm event could trigger failure. The second scenario investigates small antecedent groundwater levels prior to the main storm event. As shown in Figure 3-2b, antecedent groundwater level is modeled via an applied flux boundary condition to generate a pre-existing groundwater flow regime in the slope. In each of the remaining scenarios the level of antecedent groundwater flux is slowly increased prior to rainfall (e.g. Figure 3-2c) until the final model is brought to failure by groundwater seepage alone.

3.2.2 Landslide Model
The five physical model scenarios studied in this paper were conducted using the 5.5 m beam centrifuge at C-CORE in St. John's Newfoundland, Canada. The test configuration was chosen to model a shallow soil layer on top of a low permeability soil-bedrock contact. In the physical model, this was represented by a 50 mm thick layer of loose sand inclined at 30° (Figure 3-3). When subjected to a centrifugal acceleration of 30 g, this corresponds to a 1.5 m thick soil layer. The objective of the model was to create a triggering point where, due to a change in hydraulic gradient, groundwater would preferentially collect within the slope. This was achieved in the model geometry by continuing the soil layer beyond the toe of the slope, causing the thickness of seepage flow in this area to increase and an area of high saturation to be created in the soil model – one of the requisite conditions for the triggering of high velocity landslides under the mechanism of static liquefaction.

3.2.3 Applied Rainfall Boundary Condition
Water mist nozzles were installed inside the model package along the lid of the box to model rainfall conditions within the physical model (Figure 3-3). The nozzles were located to provide
rainfall infiltration to both the slope and the base. The nozzles were placed in a configuration such that the spray pattern fully overlapped the soil surface on both the slope and the base of the model. The resulting rainfall intensity rate at model scale was 265 mm/hr. The appropriate scaling laws for elevated gravity testing have been thoroughly described by Garnier et al. (2007) and Tamate et al (2012). The rainfall intensity at model scale is N times the amount of actual rainfall infiltration at 1g. Therefore, at the centrifuge acceleration of 30g, the applied model scale rainfall intensity of 265 mm/hr represents a prototype scale rainfall infiltration of 8.8 mm/hr.

Only one rainfall intensity rate was used in all scenarios as the objective of the test scenarios was to investigate the effect of different antecedent groundwater conditions under landslides triggered by identical rainfall events.

3.2.4 Antecedent Groundwater Conditions

Groundwater flow in the model was chosen to replicate the conditions that would occur in a loose, shallow, granular slope where seepage due to antecedent rainfall or anthropogenic sources is allowed to pool at the soil bedrock contact. The groundwater flow represents the antecedent rainfall from a wider catchment area which due to space constraints could not be physically included in the model. The groundwater flow was introduced in to the model using a metered voltage pump where an increase in input voltage increased the flow in the model ($Q_{IN}$). A buried weir at the top of the model slope (Figure 3-3) machined to the curved acceleration field of the centrifuge allowed the groundwater flow to be distributed evenly across the width of the model. The toe drain collected the groundwater ($Q_{OUT}$) into a settling tank, preventing spillage during the test and enabling the water to be re-circulated (Figure 3-3). The groundwater flow was increased incrementally in steps with each flow rate permitted to approach steady state (based on porewater pressure readings) before a further increase of groundwater was introduced. After the model reached steady state at the desired antecedent rainfall condition, the rainfall system was turned on and the rainfall event was maintained at a constant intensity until a landslide occurred.
3.2.5 Soil Layer

Davoodi et al (2010) has shown that it is critical to match the permeability of the test soil with the apparatus providing the boundary conditions in order to successfully trigger landslides in the centrifuge laboratory under controlled conditions. Out of the five test soils evaluated by Davoodi et al (2010), F110 Ottawa sand was selected as the optimal test material as its hydraulic conductivity best matched the range of controllable hydraulic flow rates provided by the metering groundwater flow pump. The properties of F110 Ottawa sand are presented in Table 2-1. The saturated permeability of the sand was reported by Take and Beddoe (2014) to be $1 \times 10^{-4}$ m/s. Following the technique of Davoodi (2008) the sand was mixed to a gravimetric moisture content of 7%, inducing suctions which permit the soil layer to be built in an extremely loose state. The soil layers were built in 5 mm lifts by raining the sand through a #20 sieve at a drop height of 100 mm. The soil layers were not tamped between lifts, thereby ensuring the loose soil structure remained. The initial void ratio in the soil layer for the five physical model scenarios was $e = 1.7$. This is significantly greater than the maximum dry void ratio ($e_{\text{max}} = 0.848$) of the F110 Ottawa sand due to the matric suction within the fine sand. A decrease in void ratio occurred when the body stress of the soil particles was increased in the centrifuge during spin-up, and then again during wetting collapse as seepage flow was introduced during testing. The maximum change in void ratio (calculated on the slope of the physical models) was 0.4 which results in a final testing void ratio averaging 1.3 on the slope. It has been shown in similar tests (Take and Beddoe, 2014) that the change in void ratio is larger along the base, reaching an initial void ratio of 1.06. The reduction in void ratio on the slope and the base of the model still exceeds $e_{\text{max}}$ confirming that the soil is in a very loose state ensuring another requisite condition for liquefaction – a highly contractile soil.
3.2.6 Instrumentation

The porewater pressures within the soil layer were monitored using eight GE-Druck PDCR-81 porewater pressure transducers installed along the slope and base of the model at the soil-bedrock contact. As shown in Figure 3-3, the PPTs are densely located in the toe region to ensure measurement redundancy and to capture the anticipated location of greatest ponding from antecedent groundwater flow (as shown in Figure 3-2). It should be noted that during post-test analysis it was found that the transducers PPT3 and PPT7 experienced a malfunction throughout the test series, and therefore have been excluded from the test results.

The displacement and velocity of landslide events were quantified using the particle image velocimetry (PIV) image analysis code geoPIV (White et al., 2003). Images of the slope profile were taken through the transparent side wall of the model chamber at a resolution of 1632 x 1200 pixels using a high-speed camera (Phantom V9.0) at a frame rate of 1000 frames per second. The field of view of the camera was targeted to capture the behaviour at the toe of the model where the initiation of the landslide event was hypothesized to occur. The PIV image analysis technique tracks subsets of the image matrix through the series of recorded frames. For these subsets to be uniquely identifiable between frames, the subsets need to have sufficient image texture within the subsets (e.g. Dutton, 2010). Since the natural F110 sand had a uniform light colour, the image texture was created artificially by layering F110 sand that was dyed black along with the natural light colored F110 sand along the transparent boundary. The resulting high contrast pattern and image matrix subsets is shown in Figure 3-4. The time series displacements for the 1000 subsets were processed using the code geoPIV and filtered to remove subpixel noise using the algorithm of Wolinsky and Take (2010).
3.3 Results

3.3.1 Scenario 1 – No antecedent groundwater (Test R1)

The first experiment investigated the scenario where the soil slope was brought to failure only under the applied rainfall storm event. The only water pooled at the base of the model prior to the applied rainfall was therefore the small amount of water resulting from the drainage of the initial moisture content of the moist as-placed soil during the application of enhanced gravity. The measured pore pressures prior to rainfall are expressed as values of total head in Figure 3-5.

The response of the soil layer to rainfall application is shown as a twenty second window of the recorded porewater pressures (PWP) covering the window of time from pre-failure to post-failure in Figure 3-6. This period of time captures the change in porewater pressures over the duration of the applied rainfall event and continues until after the landslide has occurred. To standardize time between test scenarios, the time when failure initiated in each scenario is defined herein as the time of failure, \( t_f = 0 \).

The pore water pressure results for the first scenario (test R1) show that at fifteen seconds prior to failure, the applied rainfall system had not yet been turned on (Figure 3-6a), with low porewater pressures indicating a small amount of ponded water resulting from the reduction of the initial moisture content of the soil layer upon centrifuge spin-up.

In the first scenario (test R1), the rainfall system was turned on at an intensity of 265 mm/hr (model scale) to initiate the rainfall triggering event. Within 1 second of the applied rainfall system being turned on (\( t = -13 \) s in Figure 3-6a) the rising of porewater pressures reflect the introduction of rainfall infiltration into the soil layer. The PPTs along the base of the model (PPT4-PPT8) show a faster increase in PWP than the PPTs on the slope (PPT1 – PPT3), as the
rainfall infiltrates the slope draining down to pond at the soil-bedrock contact. This increase in pore water ponding was observed to gradually continue until sudden failure occurred (t_f = 0 seconds). Failure is illustrated in the pore pressures by the dramatic spike in pore water pressure arising from shear-induced pore water pressures within the loose granular soil (Figure 3-6a).

The displacement and velocity of the landslide event for scenario 1 are shown in Figure 3-6b and Figure 3-6c respectively. The maximum distance travelled of the landslide was calculated by selecting and averaging the highest two percent of subsets defined in Figure 3-4 with respect to the magnitude of landslide travel distance. Although a landslide was triggered in this zero antecedent rainfall scenario, the landslide was characterized by a small travel distance (14 mm), a velocity of 370mm/s, and a short length of travel duration (0.059s).

All values in Figure 3-6 are reported at model scale. The accepted scaling laws associated for centrifuge testing (e.g. Garnier, 2007) indicates that linear distances are reduced by a factor N in the model, the velocity of inertial events are the same in the model and prototype, which results in the time for inertial events to occur to be reduced by N in the model with respect to the equivalent prototype behaviour (i.e. full scale). Therefore, the results of the 1/30th scale model tested at an elevated acceleration field of 30g (i.e. N=30) approximate a landslide with a full-scale travel distance of 0.42 m, a maximum velocity of 0.37 m/s, and a duration of 1.77 s. The slope instability triggered under the zero antecedent rainfall scenario investigated in the present study would therefore be considered more of a small slump than a particularly dangerous landslide. It should be noted that the difference in scaling factors for diffusion and inertial processes inherent within the technique of centrifuge modelling will allow for faster dissipation of porewater pressures in the model than would otherwise occur in the field. This issue has been described in detail by Take and Bolton (2004) for the specific case of dissipation of shear-induced porewater
pressures within centrifuge model landslide events. This problem is also well-known in centrifuge earthquake liquefaction studies (e.g. Taylor, 1995) where corrective strategies to slow down rate of dissipation by factor N include an increase in fluid viscosities by factor N, or to reduce pore sizes (i.e. grain sizes) by factor √N. As water was used as the pore fluid for this model, the landslide runout and mobility results should be viewed as being more representative of a material with particle sizes a factor √N larger. However, this scaling issue is of minor importance as the objective of the paper is to compare the relative differences between the mobility of the five antecedent groundwater flow scenarios rather than model a particular prototype slope.

3.3.2 Scenario 2 – Low Antecedent Groundwater (Test RG1)

The second scenario (test RG1) in the test series investigated the conditions where a low level of antecedent groundwater flow has occurred prior to the rainfall event. The groundwater level for this antecedent rainfall scenario was modeled using an applied flow rate of 75 mL/min, a flow sufficient to significantly raise the initial water table within the soil but not cause seepage at the toe of the slope (Figure 3-5). Upon the applied rainfall event commencing, a landslide was triggered within the first three seconds, with further shear-induced porewater pressure spikes generated during failure. The maximum distance travelled was observed to be 25 mm having travelled for 0.06 seconds reaching a peak velocity of 550 mm/s (Figure 3-7).

3.3.3 Scenario 3 – Medium Antecedent Groundwater (Test RG2)

In the third scenario the groundwater pump was increased by 50 mL/min to 125 mL/min to impose a higher antecedent groundwater condition prior to rainfall with ponded groundwater levels nearing, but not reaching the toe of the slope (Figure 3-5). The pore water pressures in this scenario (at t= -15 seconds) are up to 5 kPa greater than the values observed in test RG1 (cf. Figure 3-7a and Figure 3-8a). With this higher level of ponded water, the failure event occurred within 1 second of the rainfall event being initiated. The landslide observed in Test RG2 had a
maximum travel distance of 61 mm and travelled for 0.13 seconds with a maximum velocity of 810 mm/s (Figure 3-8).

3.3.4 Scenario 4 – High Antecedent Groundwater (Test RG3)
The antecedent groundwater flow level tested in the third scenario was sufficient to almost fully saturate the base soil layer (Figure 3-5). Further increases in groundwater flow were therefore applied in much smaller increments to approach the situation where the seepage front reached the toe of the slope and the slope is in a highly precarious state. A steady state seepage rate of 150 mL/min (only a 25 mL/min increase from the third scenario) was used for the fourth scenario (Test RG3). Within one second of applying rainfall, a landslide was triggered (Figure 3-9a). The triggered landslide travelled 113 mm (Figure 3-9b) for a duration of 0.14 seconds and reached a peak velocity of 1200 mm/s (Figure 3-9c).

3.3.5 Scenario 5 – Extreme Antecedent Groundwater (Test G1)
The fifth and final scenario investigated in the experimental study was to provide another small increment of groundwater flow in an attempt to initiate failure under the action of groundwater alone. A further 25 mL/min of seepage flow was added to the model, bringing the total seepage rate to 175 mL/min. As the slope attempted to come into steady state equilibrium with this seepage rate, a landslide occurred in the absence of rainfall (Figure 3-10a). The landslide triggered in this scenario (Test G1) travelled 71 mm (Figure 3-10b) for 0.10 seconds and reached a peak velocity of 965 mm/s (Figure 3-10c).

3.4 Discussion

3.4.1 Mobility
A comparison of the mobility of the five test scenarios is presented in Figure 3-11. In Figure 3-11a, the maximum travel distance is plotted versus time for each scenario. What is apparent
from the comparison of landslides is that the two tests with the lowest antecedent groundwater flow conditions (0 mL/min and 75 mL/min) also had the lowest travel distances. Similarly, the three tests with the highest antecedent groundwater conditions had the highest maximum displacements. In general, it is observed that the travel distance of each landslide was shown to increase with higher antecedent groundwater conditions. The exception to this observation is test G1 in which the landslide was triggered by groundwater seepage alone. In this scenario, much of the landslide mass on the soil slope was in suction at the time of failure. In contrast, test RG3 (the scenario of high antecedent groundwater levels and rainfall) had lower suctions arising from rainfall infiltration. As a result, the combination of high groundwater levels and rainfall was observed to result in a higher travel distance than the landslide triggered by seepage alone.

The mobility of the landslides for the five antecedent rainfall scenarios is further investigated by comparing the velocities of the landslides (Figure 3-11b). In general, landslides triggered with higher antecedent conditions experienced higher velocities, which were sustained over a longer duration of time. This implies that the higher mobility events experienced a higher unbalanced force (and therefore higher downslope acceleration) during failure, possibly as the result of the mechanism of static liquefaction. The differences between the scenarios investigated are further reflected in the volume of the triggered landslides.

3.4.2 Volume

The volume of a landslide also plays an integral role on the consequences a failure can generate as it travels downslope. For the five antecedent rainfall scenarios investigated, vectors of total displacement have been plotted in Figure 3-13, where in profile view the vectors capture the landslides final slip-surface and total volume. From observation alone, the low antecedent rainfall tests (R1 and RG2) show that the final failure surface is still a rotational failure. The failure event did not entrain the base of the soil layer, nor has the soil in the slope above the failure been
affected. In contrast, the high antecedent rainfall scenarios (RG2, RG3, and G1) show that final failure surfaces of the landslide are deeper, and that the movement of soil along the base is the result of liquefaction.

### 3.4.3 Failure Mechanism

While every landslide has the potential for catastrophic consequences, the high velocity, large volume and long travel distances associated with events experiencing static liquefaction make these landslides particularly dangerous. For liquefaction to occur the soil must be loose (contractile), be subjected to a monotonic loading trigger, and be sufficiently saturated. In this test series, the degree of saturation in the soil layer was the liquefaction component under investigation. One indication whether static liquefaction occurred for each scenario involves analyzing the magnitude of the shear induced porewater pressures. This is commonly performed using the pore water pressure ratio $r_u$, defined in Equation 1 as the ratio of porewater pressure to the total stress in the soil layer,

$$r_u = \frac{u}{\gamma h} \quad \text{Eq (3-1)}$$

where $u$ is the porewater pressure (kPa), $\gamma$ is the soil unit weight (kN/m$^3$) and $h$ is the depth of the soil (m). As $r_u$ approaches 1, the effective confining stress in the soil is greatly reduced. This leads to the soil's shear resistance dropping significantly below the shear stress acting on the soil, enabling large unbalanced forces to accelerate downslope. The recorded PWPs during failure for the five scenarios were presented in Figure 3-6 through Figure 3-10. By evaluating the shear induced PWPs it can be determined if the $r_u$ reached unity, indicating static liquefaction.

In all scenarios the initial triggering involved a small localized failure at the toe (Take and Beddoe, 2014). The resulting shear-induced PWPs are therefore highest in the PPT localized in the saturated loose sand closest to the toe (PPT4). In all scenarios at the location of PPT4, an $r_u =$
1 was observed indicating liquefaction or close to it. In lower antecedent rainfall scenarios (R1 and RG1) however, the other PPTs on the base of the model do not see $r_u$ values reach 1 (Figure 3-14a and 13b). Therefore in test R1 and RG, although a localized area around PPT4 showed liquefaction potential, its excess PWPs were absorbed along the base and the landslides can be classified as one of a slump or small rotational slide, not liquefaction.

In contrast the higher antecedent rainfall scenarios have the entire base experiencing simultaneous liquefaction (or near liquefaction) as shown by the PWPs annotated with an ellipse in Figure 3-14. Tests RG1, RG2, and RG3 were all sufficiently saturated along the base of the model, such that when the small localized failure began at the toe of the model, the generation of shear-induced excess PWPs enabled shearing under undrained behaviour along the base.

### 3.5 Conclusion
The purpose of this study was to evaluate the hypothesis that the travel distance and velocity of a landslide triggered under high antecedent groundwater conditions is greater than scenarios with drier conditions. Using geotechnical centrifuge modeling, five identical soil slopes were used to evaluate the hypothesis under varying antecedent rainfall conditions. Five scenarios were investigated that ranged from no antecedent groundwater flow to failure being triggered under groundwater flow alone. In the first four scenarios, the landslide was triggered by an applied rainfall storm event. In the fifth scenario, the landslide was triggered under extreme groundwater conditions only.

The results from the five scenarios show that there is a distinct relationship between the mobility of a landslide and the antecedent groundwater conditions in the soil layer prior to failure. The first scenario (test R1) had no antecedent groundwater flow and a landslide was triggered from the applied rainfall alone. The landslide triggered from this scenario had a small velocity and
travel distance. Using test R1 as the benchmark, the landslide mobility results for the remaining four scenarios have been plotted in Figure 3-14a with respect to their antecedent groundwater condition. With a constant rainfall intensity for all tests, what becomes evident is the affect that antecedent rainfall has on the mobility of a landslide. As the level of antecedent moisture condition increases, the mobility of the landslide increases dramatically. While the final scenario (test G1) has a smaller velocity and travel distance than RG3, it failed prior to experiencing any applied rainfall. For a soil layer with a finite depth there will be a level of antecedent rainfall that will saturate the slope sufficiently and create failure without additional rainfall. For this model slope configuration, that level was between 150 ml/min and 175 mL/min.

The relationship between rainfall intensity and antecedent moisture conditions and its effect on slope mobility is shown in Figure 3-14b. The antecedent groundwater conditions are plotted along the x-axis, with the rainfall duration until failure plotted along the y-axis. As hypothesized in Figure 1b, scenarios where the antecedent groundwater conditions were low (R1 and RG1), the triggered landslides had low mobility results. Even with long durations of applied rainfall, the landslides were of minimal consequence. The three scenarios under higher antecedent groundwater conditions triggered landslides which induced base liquefication during shearing, and consequentially, larger mobility results.

This experimental data illustrates the important role that antecedent rainfall conditions have on both the storm event needed to trigger landslides (Figure 3-14b) as well as the potential consequences. This test data also illustrates the complexity of generating landslide early warning systems (Aleotti, 2004). Different antecedent rainfall conditions will lead to different groundwater flow levels for different catchment areas (Montgomery and Dietrich, 1994). At
different soil depths, the antecedent rainfall required for triggering landslides will differ, leading to site specific stratigraphy data to determine soil depth (Lanni, 2012).

Despite the difficulty, the data corroborates previous researchers' findings (e.g. Terlien, 1998; Jakob and Weatherly, 2003) that there is significant merit to using antecedent rainfall conditions when evaluating the potential for storm events to trigger landslides, especially when one is interested in the mobility consequences of the potential landslide.
3.6 References


Figure 3-1 Commonly used rainfall thresholds for landslide warning systems: combinations of a) rainfall intensity and duration and b) combinations of recent rainfall and antecedent rainfall observed to result in landslides. However, not all combinations may lead to the same consequences in terms of landslide mobility. In this paper it is hypothesised that the mobility of landslide events at high antecedent rainfall is potentially higher than combinations with drier initial conditions, particularly when the soil-bedrock contact is close to the soil surface.
Figure 3-2 Boundary conditions applied in physical model tests to investigate role of antecedent groundwater on resulting landslide mobility; a) rainfall on soil slope with no antecedent groundwater, b) rainfall on soil slope after small amounts of antecedent groundwater (modelled by a low groundwater flow rate, Q), and c) rainfall on slopes with higher antecedent groundwater (modelled by providing a higher flow rate).
Figure 3-3. Physical model of loose granular landslide illustrating hydraulic boundary control systems.

Figure 3-4 Field view of high-speed camera showing PIV subsets used to track landslide displacement and velocity.
Figure 3-5 Measured total head values describing imposed conditions prior to rainfall infiltration in each of the five physical model tests.
Figure 3-6 Results of no antecedent groundwater scenario (test R1) expressed in terms of a) pore pressure, b) distal reach, and c) velocity of landslide.
Figure 3-7 Results of low antecedent groundwater scenario (test RG1) expressed in terms of a) pore pressure, b) distal reach, and c) velocity of landslide
Figure 3-8 Results of medium antecedent groundwater scenario (test RG2) expressed in terms of a) pore pressure, b) distal reach, and c) velocity of landslide
Figure 3-9 Results of high antecedent groundwater scenario (test RG3) expressed in terms of a) pore pressure, b) distal reach, and c) velocity of landslide
Figure 3-10 Results of extreme antecedent groundwater scenario (test G1) expressed in terms of a) pore pressure, b) distal reach, and c) velocity of landslide
Figure 3-11 Comparison of landslide travel distance and velocity for the five antecedent groundwater scenarios.
Figure 3.12 Comparison of vectors of total landslide displacement observed in the five antecedent groundwater scenarios.
Figure 3-13 Comparison of shear-induced porewater pressures for the five antecedent groundwater scenarios.
Figure 3-14 Relationship between antecedent groundwater conditions and a) landslide consequences, and b) rainfall duration needed to trigger a landslide given the antecedent groundwater conditions.
Chapter 4

Influence of slope inclination on the triggering mobility of hydraulically-induced flowslides

4.1 Introduction

With a significant proportion of the world's population living in areas prone to landslides, determining the probability and consequence of a landslide is of extreme importance. However, the range of factors which influence the behaviour of a landslide can vary from slope geometry or soil stratigraphy to hydraulic conditions or precipitation patterns which make predictions extremely challenging (e.g. van Westen et al., 2006). To further complicate matters, predicting the travel distance requires an analysis not only of whether a landslide is triggered (often performed by limit equilibrium), but also a model of transient pore pressure response during shearing and the propagation stage where the landslide mass accelerates and travels downslope (e.g. Hungr, 2004).

Take for instance a relatively simple factor influencing the distal reach of a landslide such as slope angle. If this is first modeled using an over simplified case of a block sliding on an inclined plane, the influence of slope angle is straightforward. This idealized case is represented in Figure 4-1a and 1b, with two slope angles of twenty and thirty degrees. If both blocks in Figure 4-1 are on a frictionless inclined plane, Newton’s second law of motion (where force is equal to mass multiplied by acceleration) tells us that the same block on the steeper slope will experience a greater acceleration. Specifically for the slope angles in Figure 4-1, the acceleration ratio between the thirty degree and twenty degree incline would be 1.46, and as such, the influence of slope angle on motion of these soil blocks is clear. If this simplified case was modified to now include friction between the block and inclined plane, one could now study the influence of slope
angle on a blocks movement by asking the question ‘At what slope angle will the block begin to move’?

Expanding this to theoretical soil mechanics, Skempton and DeLory (1957) used an infinite slope model to answer this question, where the factor of safety of an infinite slope is defined as the ratio of the sum of resisting forces to the sum of the driving forces. The effect of groundwater flow in an infinite slope model is typically modeled using a modified limit equilibrium factor of safety equation (e.g. Ray et al., 2010; Montrasio et al., 2009; Ghiassian and Ghareh, 2008), which in a simplified form is,

\[
FOS = \frac{\tan \phi (\gamma_s - m \gamma_w)}{\tan \beta \gamma_s}
\]

(Eq 4-1)

where \( \phi \) = angle of internal friction of the soil (degrees), \( \beta \) = slope angle (degrees), \( m \) = ratio of saturated soil thickness to total depth of soil \( (H_w/H) \) as defined in Figure 4-2, \( \gamma_w \) = unit weight of water \( (kN/m^3) \), and \( \gamma_s \) = saturated soil unit weight \( (kN/m^3) \).

Using Equation 1 it can been seen that for the case of a dry cohesionless soil (whereby \( m = 0 \)), the slope will remain stable at slope angles which are lower than the soils internal angle of friction. When a groundwater table is introduced to the model, the maximum height of groundwater in a soil layer must change relative to the slope angle to maintain a factor of safety greater than 1 (shown graphically in Figure 4-2). What can be inferred from the models in Figure 4-2 is that steeper slopes need a lower saturated soil thickness to trigger a failure event as compared to shallower slopes. This then suggests that under the same porewater pressures, a steeper slope will fail before the shallower slope does, making it the more dangerous slope when it comes to landslide consequences.

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Under a simplified limit equilibrium framework analysis and constraining the problem to the triggering stage of a landslide only, this observation would hold. However, the reality is in fact much less clear. The influence water has on a failure event is complex (e.g. Iverson et al., 1997; Iverson, 2000; Ghiassian and Ghareh, 2008; Leroueil et al., 2009; Olivares et al., 2009; Picarelli, 2009; Shulz et al., 2009; De Blasio, 2011), and becomes increasingly complicated when defining its influence on the post-failure stage of a failure event.

Consider a slope composed of a thin layer of loose contractive soil. In the presence of water, shearing of this soil during failure will lead to shear induced pore water pressures if the pores are sufficiently saturated. These excess porewater pressures in turn induce large shear strains to develop under undrained conditions (e.g. Lade 1992; Chu et al., 2003; Wanatowski et al., 2010; Baki et al., 2012) resulting in strain-softening behaviour associated with static liquefaction. So while the infinite slope model in Equation 2 considers the influence of water for the purpose of calculating the factor of safety, the influence and complexity of the effect of water on failure behaviour quickly exceeds the framework of the limit equilibrium model from Equation 2.

Iverson (2000), and Ghiassian and Ghareh (2008) consider the possibility of liquefaction within their limit equilibrium models, but these models focus on evaluating trigger conditions for liquefaction, not liquefaction as a post-failure behaviour. As highlighted by Cascini et al., (2013), there are a number of mathematical models for a flow type landslide that model either the triggering stage (e.g. Iverson, 2000; Buscarnera and Whittle, 2012), or the post-failure and propagation stage (e.g. Hungr, 2004; McKinnon et al, 2008) but only a limited number that actually combine these stages (Cascini et al., 2013). Further development of these combined models is currently limited by the lack of landslide events which report both the conditions leading to triggering as well as the mobility of the resulting landslide event due to extreme
difficulties in observing static liquefaction events in the field (e.g. Springman et al., 2009) and in physical models (e.g. Take, 2014).

Recent laboratory physical modeling work completed by Take and Beddoe (2014) highlight how post-failure static liquefaction type events for shallow granular soils are more complicated than can readily be captured in the typical laboratory configuration, an infinite slope model. They found that due to the inherent ease of which water pools along the bedrock-soil interface at the base of the slope over the more easily draining slope, that the area at the greatest risk of shear induced porewater pressures is the base of a slope rather than the sloping portion itself. Therefore, when a failure event is triggered, shearing of the loose saturated sand initiates post-failure liquefaction starting on the base rather than the slope. Which raises the question, if liquefaction is more likely to be triggered at the base rather than on the slope, would the slope angle have any role on influencing the post-failure behaviour of a flowslide?

Returning to an observation made in the infinite slope model discussion, it was shown that a steeper slope requires less water to trigger a failure event than a more shallow slope. By rephrasing this observation, it could also be said that the shallower slope is able to store more water before failure is initiated than a steeper slope, and with therefore, this additional amount of water could lead to the increased mobility consequences associated with liquefaction behaviour.

The question still then exists of which failure event will pose a greater risk; one triggered on a steep slope or one triggered on a shallower slope? In this paper, it is hypothesized that while it may require less water to trigger a landslide on a steep slope, a shallower slope may be more susceptible to base liquefaction than a steeper, more well drained slope and will be the slope angle which holds a greater risk if post failure liquefaction is triggered. This hypothesis will be
explored by triggering failure events in steep and shallow physical model experiments under conditions which have been designed to encourage post-failure liquefaction behaviour.

4.2 Physical Modelling and Material

4.2.1 Centrifuge Model and Soil Properties
Physical modelling has been successfully used to investigate a wide range of different failure processes in granular slopes (e.g. Eckersley 1990; Take et al., 2004; Ng, 2008; Olivares and Damiano, 2007; Tohari et al., 2007; Askarinejad et al., 2012; Take and Beddoe, 2014; Bryant et al., 2014; Jacobs 2014). In this study, the geotechnical centrifuge technique was used to test the influence of slope angle on post-failure liquefaction behaviour at an elevated acceleration of 30 gravities. The geometry of the physical model included a significant base length (250 mm past the toe of the slope) designed to ensure a sufficient zone of saturated soil such that the soil layer would promote and be susceptible to liquefaction (Take & Beddoe, 2014). The slope angle of the model was twenty degrees, selected to represent the shallow slope. The thirty degree models were first presented in the study by Take et al., (2014) and were used as the steep physical model tests. Other dimensions of the model and test configuration were selected to match the study by Take et al., (2014) to allow a direct comparison between the shallow and steep models.

A fine uniform sand (U.S. Silica F110 Ottawa Sand) was used as the granular slope material in the physical models. Properties of the sand have been well characterized by Santamarina and Cho (2001) and are reproduced in Table 2-1. The sand, mixed to a water content of 7 % was placed in 5 mm lifts, to a final depth of 50 mm. The soil depth was selected to represent a typical shallow slope when tested at the elevated acceleration of 30 gravities (i.e. 1.5 m soil layer at equivalent prototype scale). With no tamping of the material between lifts, the soil was placed moist at a very loose as-built initial state (void ratio, e = 1.7). This created a highly contractile
soil model, criteria needed for post-failure liquefaction. The void ratio reduction that was observed in the physical model during gravity turn-on resulted in a final pre-landslide void ratio of 1.06. This corresponds to a dry density of 964 kg/m$^3$, still significantly above the critical state line as defined by Santamarina and Cho (2001) and will therefore be contractive during a failure event.

4.3 Hydrogeological Conditions

4.3.1 Hydrogeological Boundary Conditions

In order for the deviatoric strain-softening associated with static liquefaction to occur, the soil must be contractive and sufficiently saturated to permit the generation of excess porewater pressures on shearing. The ability for soil in the field to reach a sufficient level of saturation for liquefaction is a function of several site specific parameters. A graphical representation of a hypothetical watershed is shown in Figure 4-3a. The ability for the soil to saturate in this watershed is shown in this model to be a function of number of parameters including: transient rainfall, area of the catchment, bedrock fractures and seepage, groundwater flow, surface storage, and precipitation. In this study, rather than model the intricate hydrogeological model for a slope in a specific watershed, a model using simplified and controlled boundary conditions was chosen. This enabled triggering of a failure event while still providing accuracy and repeatability between tests. The hydrogeological model for this study (shown in Figure 4-3b) includes groundwater flow and precipitation. An applied groundwater flow was selected as the parameter to represent all contributing factors to the degree of saturation of the soil in a watershed due to its hydrogeological conditions and antecedent rainfall. Applied precipitation was separated from groundwater flow and was thus able to trigger failure events in the physical models.
4.3.2 Application of hydraulic boundary conditions

The groundwater flow in the model \( (Q_{\text{IN}}) \) was applied and controlled using a metered flow pump and discharged into a buried weir embedded at the top of the slope (Figure 4-4). Ponding of this groundwater flux in the layer of contractive soil at the base of the slope was achieved through two hydraulic conditions. The first was a no flow boundary condition between the bedrock/sand interface and along the transparent side walls of the testing chamber. The second was through the reduction of hydraulic gradient across the model base, accomplished by promoting a zero pressure head boundary condition at the end of the base in the form of a gravity drain (Figure 4-4). These two conditions, coupled with the model geometry and the soil’s hydraulic conductivity enabled ponding to occur at the base of the model when a groundwater flow was applied. The water leaving the model at the gravity drain \( (Q_{\text{OUT}}) \) then flowed to the settling tank, where it was recirculated into the system and pumped to the top of the slope again.

Precipitation was applied in the physical model using an in-flight rainfall simulator. The rainfall nozzles were positioned above the slope and base of the model (shown in Figure 4-4), set in a pattern which ensured the spray zone of the nozzles overlapped and covered the entirety of the physical model. The rainfall simulator was set to a constant precipitation rate of 265 mm/hr, which translates to 8.8 mm/hr at prototype scale.

4.3.3 Hydrogeological Test Scenarios

In this paper, it has been hypothesized that a shallower slope may be more susceptible to post-failure base liquefaction (and its related consequences) than its steeper and more easily drained model counterpart. To test this hypothesis, failure events were triggered under three different hydrogeological scenarios to analyze not only slope angle, but also how slope angle influences antecedent hydraulic conditions and their effect on post-failure flowslide behaviour.
The first scenario examined failure events which were triggered by rainfall infiltration only. In these scenarios, when the models reached their elevated acceleration of 30g the applied rainfall simulator was turned on and remained on until a failure event was triggered. The second scenario targeted failure events also triggered by rainfall infiltration; however these models were already at increased porewater pressures. The increase in porewater pressures were achieved by having applied an equal steady state groundwater flow to the models prior to introducing rainfall. The term equal refers to the equal groundwater flux applied in both the shallow and steep models as well as with respect to the phreatic surface at the toe of the models due to the applied flow. The phreatic surface of the shallow and steep physical models under an equal steady state seepage flow of 150 mL/min shown in Figure 4-5 is modeled using the software package Seep/W. The ponding in the base layer, $m$, is quantified in this paper as the ratio of the height of the water table ($H_w$) to the soil layer thickness (H). This numerical model confirms that the physical model geometry in this study sufficiently allows for ponding in the base layer, and it can be seen in Figure 4-5 that under an equal groundwater flow, $m$ at the toe of the shallow and steep model is equal.

The final hydrogeological scenario explores the post-failure flowslide behaviour for landslides trigged by high groundwater flow only. In these scenarios, the groundwater flow was increased until a failure event was triggered, which was a different flow for the different slope geometries. The hydrogeological scenarios of the seven physical models used in this study are summarized in Table 4-1, which highlights the applied rainfall and groundwater flows in each model at the time the failure event was triggered. The applied hydrogeological conditions, along with the failure events themselves were monitored using two instrumentation methods.
### Table 4-1. Applied hydraulic conditions at the time failure is triggered

<table>
<thead>
<tr>
<th>Test</th>
<th>Slope Angle</th>
<th>Rainfall Intensity (mm/hr)</th>
<th>Groundwater Flow (mm/min)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 30 – R</td>
<td>30°</td>
<td>265</td>
<td>0</td>
<td>Chapter 3</td>
</tr>
<tr>
<td>Test 30 – G</td>
<td>30°</td>
<td>0</td>
<td>160</td>
<td>&quot;</td>
</tr>
<tr>
<td>Test 30 – RG</td>
<td>30°</td>
<td>265</td>
<td>150</td>
<td>&quot;</td>
</tr>
<tr>
<td>Test 20 – R</td>
<td>20°</td>
<td>265</td>
<td>0</td>
<td>This Chapter</td>
</tr>
<tr>
<td>Test 20 – G</td>
<td>20°</td>
<td>0</td>
<td>235</td>
<td>&quot;</td>
</tr>
<tr>
<td>Test 20 – RG</td>
<td>20°</td>
<td>265</td>
<td>150</td>
<td>&quot;</td>
</tr>
<tr>
<td>Test 20 – T</td>
<td>20°</td>
<td>265</td>
<td>200</td>
<td>&quot;</td>
</tr>
</tbody>
</table>

#### 4.4 Instrumentation

Two instrumentation techniques were selected to observe, measure and quantify the triggering and post-failure behaviour of the flowslides. The first technique was selected to monitor the transient and potential shear induced porewater pressures, and the second was to evaluate and quantify the flowslide mobility.

Eight GE-Druck PDCR-81 pore pressure transducers (PPTs) were installed in the physical model to capture the applied hydraulic scenarios and any shear-induced porewater pressures (PWP) developed during the failure event. The sensors were placed flush with the model bedrock/soil layer interface, with the body of the sensor sealed within the bedrock layer (locations shown in Figure 4-4). This provided PWP readings of the soil layer while creating no restrictions to sand movement during a failure event. The spacing of the eight sensors along the bedrock/sand interface was selected to capture the presence of water from mid-height on the slope down to the midpoint of the base layer, and would capture the influence of slope angle on soil saturation. It should be noted that PPT 7 malfunctioned prior to testing and is excluded from test results.
The failure events were tracked using a high speed imaging technique. Images were taken through the transparent sidewall of the plane-strain centrifuge model box using a Phantom V9.0 high-speed camera at a frame rate of 1000 frames per second. The dimensions of each image were 1632 x 1200 pixels, corresponding to a field of view (FOV) of 345 mm x 254 mm (the location of which is shown in Figure 4-4). The images were then analyzed using a specific Particle Image Velocimetry (PIV) method developed by White et al. (2003) to evaluate the failure events and any post-failure behaviour. This specific PIV method, GeoPIV, has been widely used in geotechnical centrifuge applications (e.g. Take et al., 2003; Lee et al., 2008; Jacobs, 2014; Take and Beddoe, 2014) and further details describing the technique for this model configuration and image analysis can be found in Take and Beddoe (2014).

4.5 Post-Failure behaviour of flow slide events

Though not every landslide transitions into a flow slide; the high velocity, large volume and long travel distances associated with events experiencing static liquefaction make these landslides particularly dangerous. For liquefaction to occur the soil must be loose (contractile), be subjected to a monotonic loading trigger, and be sufficiently saturated. In the study by Take et al., (2014), five identical thirty degree slope models with a shallow depth to bedrock were subjected to different antecedent hydraulic conditions prior to triggering failure. The results of this study showed that post-failure liquefaction behaviour can be seen using a steep model configuration (slope of 30°). In this section, a typical test result from a twenty degree model configuration will be shown to evaluate if a shallower slope can also trigger post-failure liquefaction behaviour.

4.5.1 Liquefaction in a twenty degree physical model

The increased porewater pressures for the twenty degree trial test (Test 20-T) were created by a groundwater flow of 200 mL/min. Once the test reached its in-flight acceleration of 30g, the groundwater flow was introduced to the model in incrementally increasing steps. When the PPTs
showed the model approaching steady state seepage at the final flow of 200 mL/min, the applied rainfall system was turned on which in turn triggered a failure event in Test 20-T.

The data collected during the failure event was then analyzed to evaluate if the shallower model was able to trigger a post-failure flowslide. GeoPIV software was used first to analyze the images taken during the failure event which produced total displacement vectors of the landslide. By plotting these displacement vectors at incremental time periods, the evolution of the failure event can be discerned. Figure 4-6 shows total displacement vectors for Test 20-T over 4 incremental time periods. The first time step (I) is at a time of 0.025 seconds, where the failure event has just begun and the initial slip surface on the slope of the model is visible. The second time step, II (0.06 seconds) shows a more defined slip surface that has started to reach the base of the model. The displacement vectors at 0.10 seconds (time step III) show a failure event that has transitioned from a rotational failure to a flow event. The travel direction of the displacement vectors is horizontal and flowing across the base. The final time step at 0.18 seconds (step IV) is when the landslide event had terminated and highlights the final slip surface of the failure event.

The extent of mobility from the failure event in Test 20-T can be further examined by plotting its maximum distal reach and velocity. Figure 4-7a highlights the maximum distal reach and velocity of Test 20-T, which includes time labels on the displacement curve which correspond to the time steps in Figure 4-6. The maximum displacement of the landslide in Test 20-T is 120mm, and the maximum velocity reached is 968 mm/s. Though the mobility results in Figures 6 and 7a confirm a failure event occurring, it is the shear and volumetric strain plotted in Figure 4-7b that provides the final piece of evidence that the post-failure behaviour of the landslide was liquefaction. What can be seen in Figure 4-7b is that for almost the first half of the failure event, and specifically during the period when the landslide is accelerating, there is almost no
volumetric strain. The excess shear-induced porewater pressures that are being generated aren't able to dissipate and the model is experiencing undrained shearing. The result of which is liquefaction. Once the soil starts dissipating the excess porewater pressures the soil is then able to start contracting and the landslide begins to decelerate.

Similar to the liquefaction potential in the 30° tests presented by Take et al., (2014), these results indicate that a shallower slope can also trigger failure events that transition to liquefaction during the post-failure stage. Having confirmed this, it is now possible to explore the hypothesis that a shallower slope can in fact be more dangerous than a steeper slope within a liquefaction context.

4.5.2 Mobility results from the failure events in the shallow and steep physical models

Though it is recognized that critical consequences of a failure event can vary widely depending on the context, in this study the consequences that were selected for post-failure comparison between the shallow and steep physical models were distal reach and velocity. In order to ensure accurate comparison of these consequences for tests with different slope inclinations, the same initial location for a single 64 pixel by 64 pixel subset was selected to compare distal reach and velocity. The location of the subset was chosen for its unique ability to capture the varying m ratio zone (H_w/H) under different hydraulic conditions at the toe, while still being within the area where the model is most likely to transition from a rotational failure to its post-failure liquefaction behaviour (Take and Beddoe, 2014).

In total, seven physical models were used in this centrifuge test series. The first test was presented in the previous section, Test 20-T, and was used to evaluate the ability for a shallow slope to trigger post-failure liquefaction behaviour. Of the six remaining physical model tests, three had 30° slopes and three had 20° slopes. The 30° physical models were first presented in the study performed by Take et al., (2014), where five tests were conducted to evaluate the effect
of antecedent hydraulic conditions on slope mobility and liquefaction. Table 4-2 summarizes the mobility results for all seven tests, specifically highlighting the maximum distal reach and velocity of the failure event in each tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum Displacement (mm)</th>
<th>Maximum Velocity (mm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 30 – R</td>
<td>12</td>
<td>360</td>
</tr>
<tr>
<td>Test 30 – G</td>
<td>60</td>
<td>794</td>
</tr>
<tr>
<td>Test 30 – RG</td>
<td>91</td>
<td>915</td>
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<td>560</td>
</tr>
<tr>
<td>Test 20 – G</td>
<td>121</td>
<td>1353</td>
</tr>
<tr>
<td>Test 20 – RG</td>
<td>65</td>
<td>845</td>
</tr>
<tr>
<td>Test 20 – T</td>
<td>120</td>
<td>968</td>
</tr>
</tbody>
</table>

It should be noted that there are scaling laws associated with geotechnical centrifuge testing that affect landslide mobility. Take and Bolton (2004) describe the conflicting scaling laws associated with inertial events and diffusion processes in centrifuge modelling, highlighting that by using water as the pore fluid, shear-induced porewater pressures dissipate faster than they would in cases at field-scale. This results in a quicker return to stability for physical models in a centrifuge, and subsequently smaller distal reach and velocity results than would occur at 1g field conditions. Therefore the mobility results in Table 4-2 can be used comparatively, but should not be taken as absolute.
4.6 Discussion the influence of slope angle on post-failure behaviour

Using the observed distal reach, transient velocity, and recorded porewater pressures results it is now possible to compare the consequences of failure events triggered in the physical models and evaluate whether a steeper slope is more susceptible to post-failure base liquefaction as hypothesized. The first comparison is between failure events in a shallow and steep physical model where there was no antecedent groundwater flow present, and the failure event was triggered by rainfall alone.

4.6.1 Scenario 1: Rainfall Only

The physical models brought to failure by rainfall infiltration only were Test 30-R and Test 20-R. The infiltration with respect to time from the applied precipitation (265 mm/hr) can be seen in Figure 4-8a for both tests, which shows the porewater pressures response recorded at PPT4 for the 25 seconds leading up to the failure event. In order to normalize time between test scenarios, the time when failure is initiated in each scenario is defined herein as the time of failure \( t_f = 0 \) seconds. In Test 30-R it took 13 seconds of applied rainfall to trigger a failure event, whereas Test 20-R needed 23 seconds of rainfall to trigger a landslide (Figure 4-8a). The increased period of time rainfall applied in Test 20-R permitted a greater volume of water to infiltrate the soil than as compared to Test 30-R. This increase led to marginally larger porewater pressures when failure was initiated (shown as total head in Figure 4-9a and 9b), which in turn lead to marginal differences in the distal reach (Figure 4-8b) and transient velocity results (Figure 4-8c).

However, when comparing the distal reach and transient velocities for Test 20-R and 30-R to the other five tests in this study (Table 4-2) it is clear that their failure events are significantly smaller. Because the models in Scenario 1 had no additional groundwater flow prior to failure, when a failure was triggered the soil was not sufficiently saturated for widespread liquefaction to occur. Rather, the failure events triggered in Scenario 1 would be classified as small rotational
failures or slumps that did not progress to post-failure liquefaction and result in similar small mobility (distal reach and velocity) results.

4.6.2 Scenario 2: Equal Groundwater Flow and Precipitation

The second scenario explores the hypothesis for landslides triggered in both a shallow and steep sloped physical models under medium antecedent rainfall conditions. This was achieved by first imposing an equal groundwater flow of 150 mL/min to both tests (Test 30-RG and 20-RG). The flow was applied to each model in an incremental step-wise fashion, enabling porewater pressures to approach equilibrium in the model before a further increase was applied. As highlighted by the Seep/W analysis in Figure 4-5, and shown in Figure 4-9c and 9d from the recorded PWP from the Test 30-RG and 20-RG, an equal groundwater flow regardless of slope angle results in the same Hw/H ratio at the model toe. Therefore, when the failure events were triggered for Test 30-RG and Test 20-RG there were equal zones of saturated loose soil at the toe.

When each model had reached steady state conditions at the desired flow of 150 ml/min, the applied precipitation system (at a constant rate of 265 mL/min) was turned-on until it triggered a failure event. Unlike Scenario 1 where both tests needed several seconds of precipitation to trigger a failure event, the high porewater pressures already present in the model from the groundwater flow (seen in Figure 4-9c and 9d respectively) reduced the precipitation time needed to less than a second (Figure 4-10a).

The maximum distal reach of Test 30-R and 20-R are shown in Figure 4-10b, and the velocity results are shown in Figure 4-10c. These result show that there is a nominal difference in the mobility results between the shallow and steep slopes. Though the distal reach of the slide on the steeper slope is greater than the twenty degree slope, the maximum velocities are similar. For both of these tests, when failure was triggered there was sufficient saturation for liquefaction to
occur. However, the saturation was similar enough that the post-failure liquefaction resulted in similar mobility results.

The results from Scenario 2, similar highlight that under comparable antecedent hydraulic conditions, a shallow and steep slope will have similar post-failure liquefaction behaviour. However, the hydraulic conditions in both Scenario 1 and 2 only reach a pre-determined groundwater flow (0 mL/min and 150 mL/min respectively) and the failure events required additional precipitation to be triggered. These scenarios do not maximize the shallow slopes capability for porewater pressures develop along the slope, which leads to the evaluation of the third and final scenario comparison.

4.6.3 Scenario 3: Maximum Groundwater Flow (Q)

The third scenario in this study compares the post-failure behaviour of landslides triggered in physical models that have reached their maximum hydraulic capacities. Unlike the first two scenarios which needed precipitation to trigger a failure event, the two failure events in Scenario 3 (Test 30-G and Test 20-G) were triggered by antecedent groundwater conditions alone. The flow rate which triggered a failure event in Test 30-G was 160 mL/min and in Test 20-G was 235 mL/min, and the porewater pressures these flows induce in the model are shown in Figure 4-9e and 9f respectively.

Despite the difference in the applied flow rate, the PWPs in the 25 seconds leading up to failure recorded at PPT4 are almost identical for both tests (Figure 4-11a). This is because \( m \) (the \( H_w/H \) ratio), is a function of both the flow rate and toe length. For a given model geometry there exists a groundwater flow threshold that once reached will no longer increase \( H_w/H \), even as flow rates are further increased. Simply, there is no soil at the toe left to saturate. However, as observed by the distal reach and velocity results from Test 30-G and 20-G (shown in Figure 4-11b and 11c
respectively) it is evident that while $m$ at the toe of these two tests is similar, the additional groundwater flow in the 20° tests plays a critical role on a flowslides post-failure behaviour. In fact, Test 20-G traveled 121 mm, doubling the distal reach of Test 30-G which traveled only 60 mm and almost doubled in transient velocity (1353 mm/s to 794 mm/s respectively).

The results from Scenario 3 show there are conditions in which a shallower slope will trigger a flowslide with greater consequences (quantified as distal reach and velocity) than a steeper slope, yet the important question remains of why the shallow slope traveled twice the distance and at twice the speed as the steeper slope in Scenario 3.

The answer to this lies in how the additional groundwater flow influences the post-failure behaviour of the flowslides. If a bathtub analogy were used to represent the hydraulic capacity of the two physical models, both bathtubs had to be completely filled in order to trigger a landslide in Scenario 3. However, the bathtubs in these two physical models were not the same size. There is a difference in the hydraulic capacity, or ‘bathtub size', of the 20 and 30 degree tests which in turn means the quantity of water in the physical models at failure is different. Therefore, while $m$ at the toe of these two tests is similar (and therefore the PWP's prior to failure are similar in Figure 4-11a) there is a substantial increase in saturated soil on the slope of the shallower model (Figure 4-9f) when failure is initiated than on the steeper model (Figure 4-9e). This in turn leads to an additional amount of soil that will experience undrained shearing behaviour when the shear-induced porewater pressures are generated.

This increase in undrained shearing behaviour can be seen graphically in Figure 4-12a and 12b, where the shear-induced porewater pressure spikes generated during failure are plotted for Test 30-G and 20-G respectively. In addition to the porewater pressures, the porewater pressure ratio
\( (r_u) \) has been included as a benchmark in Figure 4-11. This ratio compares the porewater pressure \((u)\) to the total stress in the soil at a given location. Thus, as \( r_u \) approaches 1, the effective confining stress in the soil is greatly reduced leading the soil's shear resistance to drop below the shear stress acting on the soil and enabling large unbalanced forces to accelerate downslope. It is therefore used as a useful benchmark when discussing post-failure liquefaction, and in Figure 4-12a and 12b has been plotted where it is equal to 1 (which is equivalent to 21 kPa for these physical model tests).

Evident from Figure 4-12a and 12b is that both tests record excess shear-induced PWPs indicative of post-failure liquefaction behaviour (the \( r_u \) benchmark of 1), however the differences are evident. In Figure 4-12a, Test 30-G had only two locations where porewater pressures exceeded 21 kPa, located where the slope and base meet (PPT3 and 4). In comparison, the toe of the slope and the entire base in Test 20-G (Figure 4-12b) exceeded 21 kPa, and did so for a significantly longer period of time than they did in Test 30-G. Plotting the peak shear-induced porewater pressures of both tests on the soil profile the extent of excess porewater pressures across the base in Test 20-G is further emphasized (Figure 4-13).

Evident from Figure 4-12, the generation of excess shear-induced porewater pressures enabled shearing under undrained behaviour in both tests. However, the additional quantity of saturated soil on the slope in Test 20-G led to the undrained shearing behaviour lasting longer and occurring throughout the entire base of the model. Therefore, while \( m \) at the toe of these two tests was similar, the substantial increase in saturated soil on the slope of the shallow model led to a significant increase in the post-failure consequences of its failure event.
4.7 Conclusions

The infinite slope model (Figure 4-2) indicates that steeper slopes need less pore water pressure to trigger a failure event than shallower slope and therefore have a higher probability of failing under the same groundwater conditions. However this also meant that a shallower slope is able to store more water and is therefore at a higher degree of saturation when failure is initiated. It was hypothesised that while a steeper slope requires less water to trigger a landslide, a shallower slope may be more susceptible to base liquefaction and will therefore hold a greater risk due to the catastrophic consequences associated with liquefaction behaviour.

This hypothesis was tested using geotechnical centrifuge physical models experiments, where failure events were triggered in steep and shallow models under conditions designed to encourage post-failure liquefaction behaviour. The results from the failure events triggered in the seven tests only further emphasized the complex interaction water has on failure behaviour and what role slope angle plays to influence that interaction. The physical models were tested under three different hydrogeological conditions, selected to encapsulate a range of ponding water in the base component of the model and the location most likely to liquefy first. In the first Scenario, where the degree of soil saturation was dictated by rainfall infiltration alone, the failure events triggered in both the shallow and steep model were small.

The second scenario explored the influence of slope angle when the physical models were subject to equal antecedent hydraulic conditions. It was found in this scenario that when precipitation triggered failure events in models subjected to an equal groundwater flow, there was a marginal difference in distal reach and transient velocity of the triggered landslides but that both experienced similar post-failure liquefaction behaviour. In contrast, when failure events were triggered by groundwater flow alone, the difference in groundwater flows needed to trigger
failure events in the physical models had a significant influence on the landslide consequences. The increased groundwater flow in the shallow slope led to an increased zone of elevated porewater pressures on the slope, which in turn enabled the excess shear induced porewater pressures to trigger liquefaction across a larger volume of soil and for a longer period of time than what liquefied in the steeper model.

Previous work has shown that modelling the effect of groundwater flow and antecedent rainfall conditions during failure is an iterative process that has a domino effect on the subsequent failure stages (e.g. Iverson, 2000; Iverson, 2005; Schulz et al., 2009). As demonstrated in this physical model study, even a simple factor such as slope angle can greatly affect what influence water has on the post-failure behaviour of a flowslide. And unlike the simplified hydraulic parameters and model geometry used in this study (Figure 4-4b), the reality is that the watershed of a slope in the field is highly complex and will only further complicate the effect of antecedent hydraulic conditions on landslides (e.g. Aleotti, 2004; Wu and Chen, 2009; Ray et al., 2010). Though a multifaceted problem, this study highlights the important role slope angle can have on the catastrophic consequences that occur when post-failure liquefaction is triggered, and adds to the database of physical model landslide events that report both the conditions leading to triggering as well as the mobility results which can be used to further validate combined flowslide mathematical models.
4.8 References


Figure 4-1. A simple block model on a frictionless incline plane where, a) is a 20° inclined plane and b) a 30° incline plane.
Figure 4-2. Infinite slope model at failure, where if all other constraints remain equal (e.g. $\gamma = 18 \text{kN/m}^3$, $\phi = 31^\circ$ and $H = 1 \text{m}$) the ratio of slope angle ($\beta$) to the internal soil friction angle ($\phi$) will influence the maximum groundwater height ($H_w$) in the model for a factor of safety equal to 1. Where the ratio and groundwater height when (a) $\beta_a = \phi$ is $H_w = 0$, (b) $\beta_b = \phi * 0.75$ is $H_w = 0.53 * H$, and (c) $\beta_c = \phi * 0.50$ is $H_w = H$. 
Hydraulic boundary conditions which influence the height of water ($H_w$) that can pond at the bedrock/soil interface for a) a typical watershed runoff system, and b) a physical model test where the simplified boundary conditions are only precipitation, $R$, and antecedent rainfall (modelled by providing a groundwater flow rate, $G$).
Figure 4-4. Geometry of the physical model illustrating imposed hydrological boundary conditions (rainfall nozzles, groundwater seepage), locations of pore pressure transducers (PPTs 1 through 8) and the high-speed camera field of view (adapted from Chapter 2)
Figure 4-5 Porewater pressure response for Test 30-Rg and Test 20-RG under the same antecedent rainfall condition (flow of 150 mL/min) where a) is the total head values from Seep/W analysis (where \( H_w \) is the height of water, and \( H \) is the height of soil) and b) is the porewater pressures at the base of the models for both tests.
Figure 4-6 Vectors showing total displacement during the failure event of Test 20-T, highlighting the landslides evolution from a small localised failure at the toe (step I) progressing into a larger flowslide (step III & IV).
Figure 4-7 The failure event in Test 20-T, where a) is the maximum distal reach and velocity of the flowslide (with corresponding time step labels from Figure 4-6), and b) is the shear and volumetric strain.
Figure 4-8 Comparison of Scenario 1 failure events (Test 30-R and Test 20-R) expressed in terms of a) pore pressures at PPT4 (see Figure 4-4 for location on model), b) distal reach, and c) velocity.
Figure 4-9 Total heads recorded at PPTs at the onset of the failure events for tests in Scenario 1 (a and b), Scenario 2 (c and d) and Scenario 3 (e and f). Figure 4-9c through 9f also include the modelled total head values for the applied groundwater flow from Seep/W analysis (dashed line).
Figure 4-10 Comparison of Scenario 2 failure events (Test 30-RG and Test 20-RG) expressed in terms of a) pore pressures at PPT4, b) distal reach, and c) velocity.
Figure 4-11. Comparison of Scenario 3 failure events (Test 30-G and Test 20-G) expressed in terms of a) pore pressures at PPT4, b) distal reach and c) velocity.
Figure 4-12 Comparison of shear-induced porewater pressures in Scenario 3 for a) Test 30-G (flow of 160 mL/m), and b) Test 20-G (flow of 235 mL/m). The time of the peak shear-induced porewater pressures (labelled A-A’ and B-B’ respectively) is the snapshot in time for the shear-induced porewater pressures plotted in Figure 4-13.
Figure 4-13 Peak total heads recorded during a) Test 30-G at time A-A' on Figure 4-12, and b) Test 20-G at time B-B' (see Figure 4-12 for PPT symbol legend)
Chapter 5

Challenges associated with physical modelling of suction dominated processes: Comparison of reduced-scale and enhanced gravity testing of landslides triggered under transient seepage

5.1 Introduction

Landslides represent a significant hazard to life, livelihood, buildings and linear infrastructure. The often complex nature of these events makes the prediction of triggering and travel distance of potential landslide debris highly challenging for the geotechnical engineering community. For example, the typical response of a landslide triggered under rainfall infiltration is a drained shearing event, where the gradual rise in transient pore water pressures leads to the onset of failure and a slip of the unstable soil until the slope angle is reduced and the mass regains its stability. However, Chapters 2 – Chapters 4 in this thesis have shown that in certain circumstances this initial drained failure can transition into an undrained shearing event, resulting in a flowslide (or static liquefaction) in loose, saturated, granular soils. Further, while the loss of shear strength associated with the undrained instability may last for only a short duration of time, it was shown that a significant drop in shear strength can be enough to subject the landslide mass to sufficient accelerations which significantly increase the velocity and distal reach of the landslide debris.

The catastrophic consequences associated with landslides dictate that predictions regarding the triggering and subsequent behaviour of a potential failure event need to be made with as much certainty as possible. Often, runout and stability forecast models are used to predict these events, examples of which are DAN-W (Hungr, 1995; McKinnon et al., 2008) and CHASM (Finley et al., 1999 and Thiebes et al., 2013). However, these models rely on rheological model parameters
obtained through back-analysis of actual flowslides to be validated. And yet the inherent nature of landslides is that they occur unexpectedly and are distinguished by rapid movement, which makes them extremely difficult to observe in the field (e.g. Springman et al., 2009). This then presents a challenge analogous to the "chicken and the egg" scenario. In order to accurately predict the triggering or behaviour of a landslide, one needs to observe landslides in action. An act which is difficult to do without accurate prediction tools. Overcoming this unpredictability, and other challenges associated with observing landslides in the field (e.g. uncertain soil stratigraphy, irreproducible or unknown boundary conditions) has led to the significant use of physical modelling techniques, which often fall under either flume tests at 1g or enhanced gravity (centrifuge tests). The appropriate testing methodology depends on the landslide phenomena being investigated.

For example, flume tests on the distal reach of dry granular avalanches is one case where small scale models provide consistent answers. Using a small scale model, Bryant et al. (2014a) found that so long as the volume of source material was large enough to ensure there was a sufficient number of particles to flow like a granular liquid rather than a granular gas, a consistent set of frictional parameters could be used to model scenarios of different volumes with a frictional rheology with DAN-W.

If particle breakage is an essential component of behaviour, such as in studies of fragmentation and spreading of rock avalanches (e.g. Bowman et al., 2012), the use of a centrifuge is advantageous as it increases the force of impact achievable in small scale models. However, as a caveat, the flow of resulting debris needs to be interpreted within the framework of the additional Coriolis acceleration acting in the centrifuge model (e.g. Schofield, 1980; Bryant et al., 2014b).
Landslide phenomena in saturated soils, such as the investigation of flowslides in submerged dyke tests by de Groot et al. (2012) are also well suited to flume testing. One advantage of flume tests in this scenario is that the failure events occur at a slower rate than centrifuge tests making requirements for instrumentation less onerous in terms of miniaturization and temporal resolution. In addition, there is typically access to the model during tests for sampling and observation, whereas these tasks need to be performed remotely in elevated gravity tests. However, flume tests often require a large quantity of soil resulting in challenges associated with material handling, as well as the long test turnaround cycle and the associated logistical constraints on the number of tests that could be performed within a given investigation. Similar tests can be done in the centrifuge, however in a centrifuge model at an enhanced gravity, N, there are conflicting scaling laws with respect to diffusive and dynamic time that create challenges in geotechnical centrifuge modelling (e.g. Take et al., 2004; Garnier et al., 2007). Dynamic time has a scaling factor equal to 1/N, whereas diffusion has a time scale factor equal to 1/N^2 which in a landslide event, results in rapid dissipation of porewater pressures in the model (t_{model dissipation} \* N^2). Resolution of this conflict requires either increasing the viscosity of the model fluid or decreasing the particle size of the soil as described by Take et al. (2004).

However, the investigation of rainfall and groundwater induced failure inherently involve transient seepage through unsaturated soils (e.g. Ng and Shi, 1998; Cascini et al., 2010; SORBINO and NICOTERA, 2013). If a small-scale model is used, capillary effects can dominate the stress conditions, similar to the behaviour one sees in a sand castle. Centrifuge testing therefore provides a significant advantage for the modelling of this phenomena despite the conflicting scaling laws for inertial and diffusion effects. The soil quantity and material handling are significantly reduced, there is short period of turnaround time between tests, boundary conditions are clearly defined, and the prototype scale can be significantly larger than what could
realistically be tested in a laboratory flume. However, these advantages are not always recognized in the literature with many case studies still being reported of small scale tests for this type of landslide failure. As such, failures in these models almost always happen as small seepage erosions at the toe with a large unsupported face which does not represent the failure behaviour seen in centrifuge tests or field events. A clear demonstration of this point would be a useful addition to the literature on rainfall and groundwater induced landslides.

A large-scale flume facility has recently been constructed at Queen's University, designed to model a number of slope stability and debris-flow events. This therefore provides a unique opportunity to provide a clear demonstration of the impact of suction in small-scale models by using nominally identical soils in a reduced-scale flume and centrifuge test. The objective of this chapter is to make the first ever direct comparison of a reduced-scale and centrifuge model using nominally identical soils, and while using this small flume event, to test and evaluate the monitoring and boundary condition control systems in the new flume on a soil layer that took weeks to construct rather than months.

5.2 Experimental Setup and Methodology

5.2.1 Queen’s University Landslide Flume

The Queen’s University Landslide Flume was designed to study a wide range of landslides and mobility behaviour (e.g. Bryant, 2013). The experimental flume (shown in Figure 5-2) is a 2.09 m wide channel with an 8.23 m sloped section, and a 36 m horizontal runout section. The angle of the sloped section is adjustable, however it remained fixed at 30 degrees to the horizontal in this study. The flume’s transparent walls are 19 mm thick tempered glass panels, extending 3.68 m in length along the horizontal section. The glass walls are fixed within the base of the flume and deformations from soil loads are further reduced with bracing at the top of the panels. The thickness of the aluminum base floor is 25.4 mm with reinforcing stainless steel joists to increase
flexural stiffness. The basal surface of the channel is the smooth aluminum, therefore a frictional coating (a textured paint) was applied to increase the interface friction angle between the soil and channel.

5.2.2 Soil Material and Preparation
The novel approach of comparing landslide behaviour in a centrifuge and reduced-scale flume test meant that the soil selected for the reduced-scale flume was chosen to have similar grain size profile as the centrifuge model. Therefore, the soil chosen for this study was a fine grained #730 Silica Sand by Weldon Company. The grain size distribution for it and the Ottawa F110 silica sand used in the centrifuge (shown in Figure 5-3) show that the soils have almost matching grain size distribution curves and for the purpose of this study, can be modelled as the same soil.

The #730 Silica sand was delivered in bulk to the experimental landslide facility (Figure 5-4a), arriving with a gravitational moisture content of less than 1%. The sand was then stored in the horizontal runout section of the flume, 20 metres away from the sloped portion of the flume where it had reached a moisture content of 5% at the time of model construction. The first step in the construction of the model was transporting the large quantity of sand (over 3.5 cubic metres) from its storage location to the test location. This was accomplished by placing a shovel full of sand at a time onto a conveyor belt, which would then travel the 20 m distance on a configuration of conveyor belts to the other end (Figure 5-4b through f). When the material reached the area in the flume where the model was being constructed, it was pluviated by hand in lifts along the slope and base (Figure 5-5a and b). This modified wet pluviation technique, done from a variable height with no soil tamping was found to be the most consistent method for creating the contractive soil layer in the model given the quantity of sand used and length of time needed to create the model. This modified method created a very loose as-built initial state in the soil layer of the model with a \( \rho_d = 1010 \text{ kg/m}^3 \) and a void ratio equal to 1.7. Steps taken to ensure
accuracy in the thickness in the soil layer included outlines of soil height on the transparent walls and frequent measurements with depth rulers (Figure 5-5c).

### 5.2.3 Application of hydraulic boundaries

Comparing landslide behaviour in the centrifuge and reduced-scale test triggered by the same mechanisms required hydraulic conditions equal to those in the centrifuge test. Therefore, the flume facility was equipped with two hydraulic boundary systems for the application of rainfall in the physical model. The first system was designed to model antecedent rainfall conditions present in the slope prior to a landslide being triggered and the second system was built to apply a precipitation event to the model.

To simplify the effects of modelling antecedent rainfall conditions in the flume, the condition was modelled through the application of a groundwater flow, entering the model at the head of the slope. Although groundwater flow in the field would be influenced not only by antecedent rainfall, but also hydrogeological and human influences, in this study those variables are constant and the groundwater flow is defined as modelling only the effect of antecedent rainfall conditions. The groundwater flow was introduced to the model through an outlet at the top of the flume, where it infiltrated the soil via an embedded weir (Figure 5-2) designed to establish uniform infiltration across the width of the flume. A weir in the floor of the flume channel was built 15 m from the flume slope in order to collect the water, and acted as a small settling basin allowing sand particles suspended in the out flowing water to settle before being pumped (Figure 5-6a) to a water reservoir.

The second system was an artificial rainfall-simulator designed to create precipitation events over the model in the flume. The system was elevated along the cross braces (shown in Figure 2a and Figure 5-6b), and was controlled separately from the groundwater system. The artificial
precipitation was created using FullJet 1/4HH-10SQ nozzles, with a flow rate limited to 32 mL/s per nozzle. There were 27 nozzles in total, set in 9 rows of 3 nozzles each. The flow between each nozzle was calibrated to within ± 2 mL/s, which evenly distributed rainfall across the flume, and the spray pattern from each nozzle overlapped to ensure complete precipitation coverage on the physical model (shown in Figure 5-2b).

5.2.4 Applied hydraulic scenario
The hydraulic scenario selected for the reduced-scale tests was chosen to match the conditions applied in the centrifuge test presented in Chapter 2. In that test, the triggering mechanism of the landslide was groundwater flow. Therefore, it was intended that the reduced-scale test would be subjected to a groundwater flow that would initiate a failure event. Similar to the centrifuge test, this was done in stepwise fashion. The first imposed groundwater flow was applied over a period of hours, after which a long period of no flow conditions was applied. A second pulse of groundwater flow was then applied, immediately followed by a third, and increased groundwater flow which triggered a failure event. Complete details of the hydraulic scenario and geometry of the physical model are summarized in Table 5-1, and are further presented in Section 5.4.1.

Table 5-1. Reduced-scale soil thickness and applied hydraulic scenarios

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<th>Geometry</th>
<th>Base (m)</th>
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<td>Triggering Groundwater Pulse</td>
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<td>0.17</td>
</tr>
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</table>
5.3 Instrumentation

Centrifuge tests have shown the importance of measuring imposed hydraulic conditions and shear-induced porewater pressures for studying triggering mechanisms and flowslide behaviour. In the small scale centrifuge models, space and miniaturization costs limit the quantity of porewater pressure transducers that can be installed. In comparison, using a flume provides ample space for sensor installation which are a fraction of the cost of the miniature centrifuge sensors. As such, 30 miniature porewater pressure transducers were installed in the flume. The MPX5050 Freescale piezoresistive transducers (an example of which is shown in Figure 5-6c) measured gauge pressure across a 0 to 50 kPa range, with a response rate of 1.0 ms. The factory calibrated linear relationship of pressure input to voltage output was validated in the laboratory prior to installation and then again after installation in the flume.

The purpose of the porewater pressure transducers (PPTs) was to record imposed and induced porewater pressures in the physical model without causing interference to potential soil movement. As such, the sensors were installed along the underside of the flume floor and had a tube attached to their port which created a flush interface with the top of the flume floor. The majority of the PPTs were located along the centre line and right edge of the model (shown in Figure 5-2b). The placement of the sensors in this configuration was to provide monitoring along the length of the flume while also capturing non-uniform groundwater flow distribution across the width of the flume. Because results from the geotechnical centrifuge studies highlighted that the toe of the model was the area where a significant portion of the excess shear-induced porewater pressures are generated, there was a redundancy in the number of PPTs in this location (Figure 5-2b).
Five synchronized high speed cameras were used to track soil movement during a failure event (Figure 5-6d). The Prosilica GX 1050 cameras captured high resolution images (1024 pixels by 1024 pixels) at a frame rate of 100 frames per second. Additional 500 watt T3 halogen lights were used to ensure sufficient light frequency for the high speed cameras. The targeted field of views for the five high speed cameras was selected to capture movement along the base and slope of the flume (Figure 5-2a). Four cameras were used to monitor the model in plane-strain and the fifth was positioned to capture the general base behaviour of the model from an elevated height. Images taken from the Cameras 1 – 4 of the physical model prior to the introduction of hydraulic conditions are shown in Figure 5-7. The 24 mm Nikkor lens coupled with the distance between the cameras and flume corresponded to field of view sizes averaging 715 mm by 715 mm.

The image processing technique of digital image correlation (DIC) uses comparisons of image frames to evaluate the displacement, velocity and strains of a failure event captured using high spatial and temporal resolution cameras. The specific DIC implementation used in this study, GeoPIV, was developed by White et al. (2003) and has been widely used in geotechnical physical modelling applications (e.g. Take et al., 2004; Lee et al., 2008; Bryant, 2013; Jacobs, 2014; Take and Beddoe, 2014; Take 2014b) details describing the technique for rainfall-induced landslides can be found in Take and Beddoe (2014). The inset of Figure 5-7 highlights the PIV subsets used in the reduced-scale test to evaluate the failure event, as well as the control markers that were used for data calibration. The zebra texture pattern visible in Figure 5-7 was created from #730 sand that was dyed black, and creates the critical contrast needed to track PIV subsets (Dutton et al., 2011). Figure 5-8 highlights the final stage of construction prior to the start of testing. The black sand pattern is added to create visible texture patterns used for PIV on images taken with Camera 5 (Figure 5-8a and Figure 5-8b). The placement of Camera 5 is at the top of the 10 m camera tripod shown in Figure 5-8c. Figure 5-8d is an image of the physical model as completed.
prior to the commencement of testing. Components visible in Figure 5-8d include the extra light sources, the artificial rainfall system and the location of the data acquisition system (bottom right corner of the image).

5.4 Results and Discussion

5.4.1 Model response to hydraulic events

The initial pulse of groundwater flow was applied to the physical model using a groundwater flow of 1.8 L/min for a period of 4 hours, shown in Figure 5-9a. The recorded porewater pressure response to the initial pulse is plotted in Figure 5-9b and Figure 5-9c as the averaged pressure head for the PPT rows along the slope and base respectively. Initially, the only PPT to register an increase in pressure head is the PPT Row S4. This PPT row is found directly above the weir, and therefore for the first 3 hours maintains a steady state seepage, from the steady state flow. The dramatic spike downward of this PPT is due to the soil eroding slightly at the weir and is therefore no longer is able to pool water above the sensor.

When the initial groundwater flow was terminated (at a time of 4 hours), an increase in pressure head had only begun to register at the base of the model along the first 2 rows of PPTs (Figure 5-9 c). It was also observed at this time that there had been preferential flow of groundwater within the soil layer. Taking a cross section along the width of the flume at PPT Row B2, the pressure head from each of the 5 sensors has been plotted in Figure 5-10a. Interestingly, images taken at the toe of the slope during the same period of time show that despite only registering an average pressure head around 5 cm in B1 and B2, there is a significant area of darkened soil above the phreatic surface. Shown in Figure 5-11a, this darkened soil layer is due to wet soil, indicating that the capillary suction of the soil layer is significantly greater than the 5 cm measured.
After the completion of the initial pulse of groundwater flow an elapsing period of time between groundwater flow conditions began. What is observed over the next several hours is that while the free flowing water from the initial event drains from the model, there are small changes of pressure head that occur in the soil layer (Figure 5-9 b and c). However, if the soil had drained as free flowing water it would have resulted in a delayed increase in pressure head as the flow traveled through the model. However, that was not observed. While there is a general decrease in pressure head from the elapsed time of 4 hours to 18 (as seen by the drop in pressure head across Row B2 in Figure 5-10 a and b), the loss is not due to water flowing across the base of the model with time but rather due to capillary suction. The capillary rise in the soil layer at this time is now almost to the height of the soil layer (shown in Figure 5-11b). Prior to the second pulse of groundwater flow starting, Figure 5-11c highlights the fact that the water within the top 10 cm of the soil layer has evaporated over the 263 hours of intermittent time, however the remaining soil layer has maintained an elevated degree of saturation due to suction.

The second pulse of groundwater flow is at a rate of 1.56 L/min. Due to the large intermittent period of time separating the antecedent rainfall events, time on the x-axis in Figure 5-9 has been truncated for easier visual analysis. While the PPTs in Figure 5-9c shown a negative pressure head, it should be noted that the sensors are not measuring suction. Rather, the height of water in the tube that attaches the sensor to be flush with flume has dropped below the top of the flume floor, which is the location of the porewater pressure datum.

With an elevated degree of saturation in the soil layer from the initial flow, it took only one hour for the soil layer at the toe of the slope to reach its previous maximum degree of saturation (Figure 5-9c). After two hours at a groundwater flow of 1.56 L/min, there is a much greater uniformity of flow across the width of the flume (Figure 5-10c). At the end of the third hour of
flow, the physical model showed signs of reaching steady state seepage (Figure 5-9b) at that groundwater flow rate. The model was held at steady state seepage for an additional 2 hours to further promote uniform steady seepage throughout the model (Figure 5-10d and Figure 5-11d).

The porewater pressure recorded in the physical model at steady state seepage has been plotted in Figure 5-12. Superimposing an inferred phreatic surface from the recorded porewater pressures on the soil layer provides further evidence that the toe and base of the model were saturated at the end of the second groundwater flow. Small irregularities within the thickness of the soil layer (either as constructed or from differential wetting collapse) permitted visual pooling of water on the soil surface which are also shown in Figure 5-10 d.

After two hours of steady state seepage, an increase to the groundwater flow was applied. A flow rate of 3 L/min was applied to the model, triggering a failure event after 10 minutes.

5.4.2 Initial Failure Event
The initial failure event which was triggered in the flume 269 hours, 10 minutes and 12 seconds after the initial groundwater pulse was initiated, lasted less than 2 seconds. The movement of the slide was captured by the synchronized high speed cameras and was evaluated using GeoPIV analysis techniques. In Figure 5-13, the total displacement vectors calculated at incremental time periods highlight the evolution of the failure event. The event has been divided into four periods of time, chosen to capture the failure event from the initial movement until the landslide has come to a complete stop. The first time step is 0.1 seconds, where the movement of the failure event up to that point is negligible and as such, the displacement vectors show as only vector dots. After 0.75 seconds have elapsed, a small localized failure on the slope has begun (Figure 5-13b). In Figure 5-13c the failure event has continued to evolve downslope, where the largest displacements are at the toe of the slope. When the failure event has come to a complete stop
Figure 5-13d), the final failure surface extends through the slope and well into the base of the model.

To further quantify the observed distal reach of failure event, the maximum displacement of the failure event calculated on the slope (within the FOV of Camera 2) and the base (Camera 3) are plotted in Figure 5-14a. The maximum distal reach of the failure event was calculated using GeoPIV analysis, which included a filtering algorithm to remove subpixel noise (Wolinsky and Take, 2010). The maximum distal reach failure event on the slope was 25 mm, and on the base was only 7.5 mm. Similar differences exist in the velocity of the maximum displacement subsets from the slope and the base, shown in Figure 5-14b.

The mobility results of this failure event highlight the landslide event triggered in the reduced-scale physical model. Further examination of the porewater pressure response during the failure event, (seen as the spike in Figure 5-9c) will conclude what, if any, transition from a small rainfall-induced rotational failure to a flowslide occurred during the failure event.

5.4.3 Porewater pressures response during initial failure event

With the triggering of the small rainfall-induced landslide, excess shear-induced porewater pressures were generated. The porewater pressures plotted in Figure 5-15 highlight pressure head in the model recorded 30 seconds prior to and after the initial failure event began. What is clear is that excess shear-induced porewater pressures are generated. However, the excess porewater pressures are starting to show signs of approaching steady state after only 30 seconds since the failure event happened. Plotted in Figure 5-15 b it can be seen that the excess porewater pressures are generated over the period of approximately one second. Similar to Eckersley (1990), it was observed that movement of the failure occurred first, which began generating the shear-induced porewater pressures. The initial failure event began first on the slope of the model,
in the area by PPT Row S2. Shown in Figure 5-15 b, this is the first row that shows signs of an increase in porewater pressures. The porewater pressure response above this (Rows S3 and S4) does not change, highlighting that the slip surface did not extend this high up the slope. Similarly, the final row of PPTs on the base (B7) did not register any shear-induced porewater pressures (Figure 5-15 b) indicating that the slip surface exited the base of the model prior to reaching it.

As presented in previous chapters, porewater pressure ratio ($r_u$) is a useful tool when describing a failure event's potential to trigger post-failure flowslide behaviours. In the reduced-scale flume model, this corresponds to a pressure head of 32 cm on average along the base. Evident in Figure 5-15a is that PPTs B1-B4 generated excess porewater pressures exceeding an $r_u$ equal to 1. However, the excess porewater pressures began dissipating after only 1 second, and the momentum of the failure event is quickly absorbed and no flowslide is generated.

5.5 Comparison of centrifuge and reduced-scale failure events

The reduced-scale and centrifuge tests were subjected to elevated groundwater flows that resulted in similar steady state seepages designed to sufficiently saturate the base of the models prior to failure. Figure 5-16 shows that this has been achieved, where Figure 5-16a is the reduced-scale model and Figure 5-16b is the centrifuge test at prototype scale. By plotting the centrifuge at prototype scale it can be seen that the scale of the two tests are equal in Figure 5-16. Both models were then subjected to an increased groundwater pulse until a failure event was triggered.

The failure event in the reduced-scale model was a small rotational failure, which quickly regained stability rather than continuing on to a catastrophic flow (Figure 5-17a). Yet under similar triggering mechanisms, the failure event triggered in the centrifuge model was a flowslide with significant mobility consequences (Figure 5-17b). These dramatically different
consequences highlight the danger of using a reduced-scale test for modelling rainfall-induced landslides, due to the critical role of suction within the models.

The centrifuge test was built with a soil depth of 50 mm, selected to represent a typical shallow slope at prototype scale. Applying geotechnical centrifuge scaling principles to account for the acceleration of 30 gravities the test was conducted at, the 50 mm soil layer represents a 1.5 m soil layer in an equivalent prototype. In comparison the reduced-scale model had soil thickness of 0.3 m. So although both models used similar fine grained sands with a capillary rise of approximately 0.5 m, the effect of this 0.5 m of suction on a 1.5 m thick soil layer is very different than on a 0.3 m soil layer.

In geotechnical centrifuge models, matric suction becomes virtually absent due to its scaling factor of 1/N, while the soil thickness increases by N (e.g. Rezzoug, et al., 2004). This is not the case for reduced-scale tests, where the 0.5 m of suction was already visible during the initial groundwater pulse (Figure 5-11). The presence of suction when the failure event was triggered in the reduced scale model played two roles. The first is the significant increase of strength due to capillary cohesion above the water table, which increases stability. The second is the inherent ability for the soil layer to dissipate excess generated porewater pressures during the failure event. This would result in the failure event that is triggered to regain its stability significantly faster than it otherwise would in a centrifuge model.

5.6 Conclusion
A new flume facility has recently been constructed at Queen's University, and the development of this flume for reduced-scale physical models has been well documented in this chapter, including soil and material handling, instrumentation techniques, and boundary condition control systems. The availability of this flume facility has provided the opportunity to make the first ever direct
comparison of a reduced-scale and centrifuge model to demonstrate the impact of suction in small-scale models. The reduced-scale and centrifuge models were built using nominally identical soils, and after being subjected to similar hydraulic conditions (Figure 5-16) were brought to failure under similar triggering mechanisms in order to isolate the impact of suction. And yet, there is a dramatic difference in the landslides triggered (Figure 5-17).

The soil in the tests were selected to be nominally identical, which resulted in both models using a soil with a capillary rise of 0.5 m. In the centrifuge test, scaling factors significantly reduce suction, and so the 1.5 m prototype shallow landslide is not impacted by a 0.5m capillary rise when triggered at elevated groundwater levels. However, in the 0.3 m thick soil layer in the reduced-scale test, it is clear that this becomes a suction-dominated problem. Not only is the slope inherently stronger due the effect of capillarity cohesion, but when a failure event is triggered the landslide is quickly able to regain its strength as porewater pressures quickly dissipate.

This demonstration, which has shown the significant impact suction plays on reduced-scale models, highlights the challenges and dangers associated with using reduced-scale models to study rainfall and groundwater induced landslides. Despite the advantages of the highly instrumented reduced-scale test, it is clear that capillarity governed the failure event and had significant impact on the consequential behaviour of the landslide. Care needs to be addressed when using a reduced-scale model regarding the choice of soil thickness and grain size due to their significant influence on suction. Options would include building a thicker soil layer to decrease the influence of suction relative to the models soil thickness, or decrease suction itself by using a coarser soil. However, these changes must then be acknowledged for the significant modifications they make to the original model intended for investigation.
This comparison of physical modelling techniques has highlighted the important role that geotechnical centrifuge modelling has when studying the triggering mechanisms and consequences of rainfall and groundwater induced landslides. Although centrifuge modelling must account for scaling laws which present challenges, there are inherent advantages that will continue to make this a particularly useful technique for the investigation of unsaturated suction-dominated problems such as rainfall-induced landslides.
5.7 References


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Figure 5-1 Effect of an extended toe on hydraulic conditions which lead to a zone of loose saturated granular material on the base. This in turn creates a zone more likely to trigger a post-failure flowslide rather than the well-drained inclined slope.
Figure 5-2 The instrumented flume where a) cross-sectional view and b) plan view. Highlighted in the figure are the porewater pressure transducer locations, camera field of views (Cameras 1 through 5), rainfall nozzle locations, and example nozzle distribution zones.
Figure 5-3 Grain size distribution for # 730 silica sand used in the flume study and the Ottawa F-110 silica sand from the geotechnical centrifuge test (Chapter 2).
Figure 5-4 Series of images showing a) the arrival of the sand, b) digging the soil from its storage location where it, c,d) is placed on a conveyor belt and travels, e,f) 20 m down the horizontal runout channel of the flume to the location where the reduced-scale model is being constructed within the flume.
Figure 5-5 Series of images highlighting a) the arrival of the sand to the location where it is being built, b) the modified wet pluviation soil placement technique and c) measuring the soil thickness during construction.
Figure 5-6 Equipment and instrumentation used in the physical model study, where a) pump used to re-circulate water flow back to storage reservoir, b) artificial rainfall system nozzles installed, c) an example of a porewater pressure transducers after it had been wired. Shown with a ruler for scale, and d) the configuration of high-speed Cameras 1 through 4
Figure 5-7  Images taken in Camera 1 through 4 imposed on flume diagram. Enlarged Camera 4 image highlights PIV subsets and control markers used to track landslide displacements. Note, Camera 5 is not included as it is taken from a height of 10 m and therefore not on the same axis as Cameras 1-4
Figure 5-8 Series of images showing the final stages of construction and instrumentation, where a) placement of horizontal black sand stripes, b) close-up view of plane-strain zebra pattern texture and horizontal stripe texture, c) location of tripod for Camera 5 (shown by arrow), which places the camera at an elevation of 10 m, and d) the final as-built reduced-scale physical model.
Figure 5-9 Hydraulic boundary conditions and model response, where a) is the applied groundwater throughout the duration of the test, b) the models response to groundwater flow on the slope (plotted as pressure head) and c) the model response of groundwater flow on the base.
Figure 5-10 Porewater pressure response (plotted as pressure head) across the width of the flume channel, taken across PPT Row - B2 (shown as B2'-B2' in Figure 5-2) where, a) after the completion of the initial groundwater pulse (elapsed time of 4 hours), b) elapsed time of 18 hours, c) 2 hours into second groundwater pulse (263 hours), and d) after 2 hours of steady state seepage during the second groundwater pulse elapsed time of 268 hours).
Figure 5-11 Series of images taken along the base of the model capturing the infiltration and saturation behaviour of the soil layer, a) after the completion of the initial antecedent rainfall event (4 hours), b) at an elapsed time of 18 hours, c) at the end of the intermittent time period of 263 hours, and d) after 2 hours of steady state seepage during the second antecedent rainfall event (elapsed time of 268 hours).
Figure 5-12 Pressure heads measured in the physical model by the PPTs, 268 hours after testing began (2 hours after reaching steady state seepage). Included is the inferred steady state phreatic surface and the location where water was visibly ponding along the base of the model.
Figure 5-13 Total displacement vectors of the landslide describing the evolution of the initial failure event, starting from a) 0.1 seconds after the first soil movement to d) the termination of the failure event after a total time of 2 seconds.
Figure 5-14 Mobility results of the failure event on the slope (Camera 2) and the base (Camera 3) where a) is maximum distal reach and b) is the maximum velocity.
Figure 5-15 Porewater pressure response of the failure event where a) plots the 30 seconds leading up to, and following the initiation of failure and b) for the 2 seconds of excess generated shear-induced porewater pressures. Time of 0 seconds is the start of the failure event.
Figure 5-16 Imposed groundwater flow that triggered failure events in a) the reduced scale flume test and b) at prototype scale of the centrifuge test.
Figure 5-17 Comparison of the rainfall-induced landslides initiated by similar triggering mechanisms in a) reduced scale flume test and b) prototype scale of geotechnical centrifuge.
Chapter 6

Conclusions

6.1 Overview
The objective of the research presented in this thesis was to investigate triggering mechanisms and the consequences that are associated with hydraulically-induced landslides in a loose granular soil using physical modelling techniques. The first phase in this research program used a small-scale geotechnical centrifuge model, and the second required the development of a new flume facility to examine landslide behavior in a reduced-scale model. This chapter presents the overall conclusions of the research objectives, providing the foundation for understanding the triggering mechanisms and consequences of rainfall-induced landslides and demonstrates the value of the research findings.

6.2 Conclusions drawn from the Research

6.2.1 Objective 1: Exploring the concept of base liquefaction
The concept of base liquefaction, a novel triggering mechanism for shear-induced failures of loose granular slopes was explored using a centrifuge model. The inclusion of a significant toe length in the physical model created a region of soil prone to liquefaction in an otherwise inherently well-drained model. High speed camera footage, image analysis techniques, and transient porewater pressure records demonstrated that it was the inclusion of the base region in the model that created a region of soil prone to liquefaction in granular soil. A simple toe failure acted as the monotonic loading trigger, shearing the loose contractile saturated sand at the base of the slope and causing liquefaction to occur in the base region.

Omission of the significant toe length in physical models reported in the literature provide an explanation for the high level of difficulty for triggering static liquefaction observed in these
previous attempts. The subsequent flowslide in the base layer causes a loss of support to the non-liquefied soil on the slope which in turn caused the slope to follow which is often the location believed to have triggered liquefaction through back-analyses. Great care must be used when trying to model liquefiable events using conventional limit equilibrium analysis due to the inherent challenges when modelling seepage conditions prior to failure and the measured final surface.

6.2.2 Objective 2: Role of antecedent groundwater conditions on landslide consequences

Five geotechnical centrifuge models were simulated to determine whether landslides triggered under varying antecedent groundwater conditions influence the consequences. The results of these five tests showed a distinct relationship between the mobility of a triggered landslide and the antecedent groundwater conditions in the soil layer prior to failure. It was shown that as the level of groundwater flow rises, so too do the consequences of the failure event.

More significantly, it was found that there existed a threshold of groundwater flow which would dictate the consequences of the triggered landslide. If a landslide was triggered below the groundwater threshold, the consequences of the failure event (represented by distal reach and velocity) were relatively small. However, when the landslide was subjected to a groundwater flow exceeding this threshold, the failure event would transition into a flowslide and therefore have much larger consequences.

The catastrophic nature of the landslides which have exceeded the groundwater threshold put significant weight behind using a "consequential threshold" when using landslide prediction tools. The use of a which would enable intensity-duration curves to start predicting not only the probability of a landslide, but also the mobility consequences improving risk analysis for predication tools.
6.2.3 Objective 3: Quantifying the influence of slope inclination on landslide consequences

Seven geotechnical centrifuge models were built and tested to compare the consequences of landslides triggered in a loose granular soil layer inclined at 20° and 30°. The results of these tests indicate that the influence of slope angle on the consequences of a triggered landslide are highly dependent on the antecedent groundwater conditions. Failure events which had been subjected to no antecedent groundwater prior to being triggered by rainfall had minor consequences irrespective of slope inclination. An increase in groundwater flow prior to the triggering event resulted in larger consequence events, however there was still only nominal differences with respect to the consequences between slope inclinations.

However in high antecedent groundwater conditions, a landslide triggered on the shallower 20° slope was observed to travel twice the distance reaching nearly twice the maximum speed as compared to a landslide triggered on the steeper 30° slope. The reason for the dramatic difference in consequences is the increased region of soil at elevated porewater pressures when failure was triggered in the shallow slope. This enabled excess shear induced porewater pressures to trigger static liquefaction across a larger volume of soil and for a longer period of time. These results provide experimental data documenting the important roles slope angle and antecedent groundwater conditions play on the consequences of post-failure flowslides in loose granular soils.

6.2.4 Objective 4: The development of a new flume facility to examine landslide behavior in a reduced-scale model

A new flume facility was developed to examine landslide behaviour in a reduced-scale model. The flume was designed to capitalize on its inherent advantages over small scale studies such as bench top flumes or centrifuge models. The base of the flume was heavily instrumented with porewater pressures transducers, and five high-speed cameras were used to capture the event.
The synchronized instrumentation systems were selected based on their high resolution and frequency of data, which enabled the 2 second event to still have a data rate of 0.01 seconds after 269 hours of testing. One noted disadvantage in the development of a flume facility is the turnover time between tests due to the large scale of material storage and handling challenges. Taking weeks to prepare, this test was only a reduced-scale model and due consideration must be made when selecting this option for future physical modelling.

**6.2.5 Objective 5: Comparison of reduced-scale and enhanced gravity testing of landslides triggered under transient seepage.**

The first ever direct comparison has been made of a reduced-scale and centrifuge model using comparable soils to demonstrate the impact of suction in small-scale models. In both models, landslides were triggered by the same mechanism of a rising groundwater table. In the reduced-scale model, the significant presence of suction above the water table enabled the failure event to quickly regain stability. In contrast, the more realistic ratio of soil thickness to suction in the centrifuge model dramatically increased the consequences of the landslide. The results of this comparison show the significant impact suction can play on the consequences of a landslide and highlights the challenges and dangers associated with using reduced-scale models to study suction-dominated, inherently unsaturated problems such as hydraulically-induced landslides.

**6.3 Impact of Research**

The physical modelling studies conducted in this research program have significantly furthered our understanding of the triggering mechanisms and consequences of hydraulically-induced landslides in loose granular soils.

The identification of base liquefaction as a mechanism for shear-induced failure of loose granular slopes provides a new mechanism that should now be considered when evaluating landslide hazards in this type of material. Rather than focusing solely on densification of loose granular
fills or drainage along the slope, significant attention should be paid to areas where groundwater can pool as these areas create soil regions prone to liquefaction. Specifically, it has been addressed that pooling of groundwater at locations where the hydraulic gradient reduces will be the more likely location to liquefy, and care must be taken to accurately locate these areas and address them. Similarly, it has been shown that in the case of significant pooling of groundwater, the inclination of the slope above can play an important role in the consequences of triggered landslide. Due care should be taken to address shallow slopes that have an area prone to liquefaction.

Evaluating different transient seepage scenarios in slope models has demonstrated that it is possible to define a threshold of seepage that is needed to trigger high consequence flowslides. This consequential threshold concept has significant impact when targeting preventative measures for landslides. The high costs typically associated with preventative measures significantly influence the degree of action that can be taken to minimize the landslide hazards. However, if the consequences of a potential landslide is known, preventative measures can then be targeted at the high risk landslide locations in order to make the greatest impact with the available resources.

The physical modelling comparison has shown the significant impact suction plays on reduced-scale models, highlighting the challenges and dangers associated with using reduced-scale models to study hydraulically-induced landslides. If reduced-scale models are used to study these failure events, care needs to be taken regarding the choice of soil thickness and grain size due to their significant influence on suction. Perhaps even more critical however is that the physical modeler understands this relationship in a reduced-scale model and addresses the influence it has on the outcome of their results. Though reduced-scale models have their place, it is highly
recommended that centrifuge modelling is used when investigating landslides in unsaturated soils.

6.4 Next Steps
This research program has yielded significant findings and results. However, the inherent nature of research is that inquiry often leads to both a deeper understanding of the problem and additional research questions. As such, a number of interesting ideas and questions have arisen from this research, and are suggested as areas of future work.

6.4.1 Centrifuge Modelling
The work reported in this thesis has focused primarily on antecedent groundwater as a variable. However, in the future, it is possible to investigate the influence of infiltration processes on triggering mechanisms through the provision of multiple rainfall events. The rapid turn-around associated with centrifuge modelling would also permit the investigation of the critical void ratio needed for base liquefaction to be triggered, and a comparison of this void ratio to that predicted within the instability line framework.

6.4.2 Flume Facility
Knowing the impact suction had on the consequences of the reduced-scale landslide in this research, the obvious next step is to conduct a similar test using a more coarsely graded sand. In this future work, the thickness of the soil layer should be maximized to further reduce the influence of suction.

6.4.3 Numerical Modelling
The scope of the research in the present study was limited to physical modelling. Significant opportunities therefore exist to use this experimental data to validate numerical modelling outcomes. With respect to the numerical modelling of triggering, unsaturated slope stability models of these failure events using Slope/W and Seep/W would provide an interesting
opportunity to explore the complex transient behaviour of unsaturated soils. The data reported in this thesis also represents a unique opportunity to investigate the combined triggering and post-failure mathematical model by Cascini et al., (2013).

6.5 References