DYNAMIC PILE-SOIL INTERACTION IN LATERALLY SPREADING SLOPES

by

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Abstract

The collapse of buildings and infrastructure is an unfortunate consequence of major earthquakes (e.g., the 1964 Alaskan earthquake, the 1995 Kobe earthquake in Japan and the 2007 Pisco earthquake in Peru). Liquefaction-induced lateral spreading is known to be one cause of severe damage to deep foundation systems. However, the dynamic soil-structure interaction between liquefied soil and piles is extremely complex and further work is required to define the appropriate design pressures and to understand the mechanisms at work.

This thesis presents the findings of an experimental program carried out using the large geotechnical centrifuge at C-CORE in St John’s Newfoundland, to investigate the mechanism of lateral spreading and its implications for dynamic soil-pile interaction. Soil and pile responses were measured using accelerometers, pore pressure transducers, and digital imaging using a high speed camera. Using these images, transient profiles of slope deformation were quantitatively measured using Particle Image Velocimetry (PIV). These tests illustrate the potential for earthquake shaking to excite the natural frequency of the liquefied soil column, which can lead to increased transient lateral pressures on piles in liquefiable ground. This study recommends that this potential for “auto tuning” should be anticipated in design and proposes a new limiting pseudo-static backbone p-y curve for use in the design of piles subjected to lateral spreading ground deformation.
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Chapter 1
Introduction

1.1 Statement of the problem

Liquefaction-induced lateral spreading is known to be one cause of severe damage to deep foundation systems installed in sloping ground. Lateral spreading typically occurs in soil slopes that are saturated, loosely-deposited, and are gently sloping. In these slopes, earthquake-induced excess pore water pressures result in a significant reduction of soil available shear strength. Because of this, these slopes are easily deformed by the existing static shearing forces and the additional dynamic inertial forces imparted by the earthquake. Any foundation systems embedded in these slopes, such as piles forming bridge foundations, will therefore be subjected to additional lateral pressures as the lateral spreading soil flows past the piles. Adding to the complexity of the problem, these piles are simultaneously subjected to a loss of lateral support (due to the liquefaction of the soil) while undergoing lateral forces from both earthquakes and lateral displaced soils.

Whereas structural failure of the pile structural element due to these increased dynamic loading is one failure criterion, excessive deformation of the pile is another. Superstructures can be harmed if the deformation of the piles exceeds the allowable displacement at the connections. Examples of this scenario include the famous Showa River bridge failure during the 1964 Niigata Earthquake (Hamada, 1986). Since we are unable to prevent the occurrence of earthquakes, research must be focused on understanding the soil-structure interaction problem describing the load-displacement relationship between pile foundations and lateral spreading soil.
1.2 Current research needs

Due to its inherent simplicity, the load-displacement relationship method based on the Beam on Nonlinear Winkler Foundation (BNWF) approach is widely used to model laterally-loaded piles. This method was initially developed to predict horizontal pile deflections under lateral loading conditions. However, in the case of lateral spreading, the displacement of the free field soil causes the pile to deform laterally. In this case, the pile displacements used in the method are therefore substituted by relative displacement of pile and soil displacement.

For the case of piles in laterally spreading soil, questions arise as to the design pressures which should be expected for liquefied flowing soil as input parameters to this and other models of lateral soil-structure interaction. Complicating matters is the pore pressure generation, phase transformation, softening, and pore pressure dissipation within the soil and the inertial loading applied to the soil-structure system. Due to the complexity of this phenomenon, physical modeling presents a unique opportunity to observe and understand the mechanisms at work behind the soil-structure interaction with a view to improve design recommendations and to calibrate subsequent numerical models. Although researchers have worked on proposing methods of analysis, supplementary data is needed to certify the reliability of the methods.

Due to the restrictions of measurement technology, all previously published experimental studies on this problem were limited to the measurement of deformation at the surface (during the earthquake), and/or the profile of soil displacements post-earthquake. In this thesis, the combination of high-speed imaging and digital image processing codes enable the profiles of transient soil deformations to be observed, and the load-displacement relationship between pile foundations and lateral spreading soil to be defined to an extent not previously possible.
1.3 Scope and objectives of thesis

This thesis discusses the results of an experimental study on lateral spreading and pile responding to lateral spreading during strong shaking. Centrifuge models were came out to replicate the scenario of piles supporting bridges where the ground is usually submerged and loosely-deposited.

The first objective of the thesis is to study the free-field mechanism of lateral spreading and to provide physical data from the perspective of evolution of soil displacement during lateral spreading. The response of soils including acceleration, pore water pressure and transient displacements are presented. The study focuses on the investigations of the attenuation and amplification of acceleration and the displacements including the in-cycle, surface, soil profile and slope profile displacement.

The second objective of the thesis is to study the interaction of laterally spreading soils and pile foundations and produce design recommendations regarding the appropriate p-y curve applicable to piles in liquefiable soils. This second study combines the observed free-field response with the measured pile response to calculate bending moments and lateral pressures imparted on the pile as a function of relative soil-pile deformation.

1.4 Organization of thesis

Chapter 1 Introduction

This chapter briefly describes the work containing the introduction to lateral spreading problem, current research needs on the topic, the purpose of the study, and the organizational summary of the thesis.
Chapter 2 Literature Review

This chapter provides background knowledge from the literature. It includes previous studies of lateral spreading concerning the mechanisms, displacement prediction and experimental modeling. The methods of predicting the lateral displacement proposed by several researchers are described as well as the experimental studies of piles in lateral spreading.

Chapter 3 Evolution of Lateral Displacement in Lateral Spreading

This chapter presents the results of centrifuge modeling observing lateral spreading. The mechanisms including soil accelerations, pore water pressures, and displacements are discussed. It details the attenuation and amplification of soil accelerations and the progression of soil lateral displacement.

Chapter 4 Piles in Lateral Spreading

This chapter details the observation of centrifuge modeling on pile behaviour during liquefaction-lateral spreading. The data concerning soil-pile interaction during lateral spreading observed from signal conditioning equipment and a high speed camera are provided as well as data concerning in-cycle and transient behaviours.

Chapter 5 Conclusions

This chapter provides a summary of the work and implication for design procedure. Recommendations for future work are also provided in this chapter.
Chapter 2
Literature Review

2.1 Liquefaction

2.1.1 General information

In common usage, liquefaction is a phenomenon that occurs when a saturated soil is subjected to rapid shear stress cycles, leading to a rapid volumetric strain decrease. If the pore fluid between soil particles does not have time to dissipate, high pore pressure reduces the friction between soil particles, the mean effective stress decreases, and the strength and stiffness of soil is reduced. Figure 2.1 (a) shows saturated soils deposited in which the contact force between each grain is large enough to keep the grains in place. Figure 2.1 (b) shows increased pore pressure that reduces the contact forces between the grains and the soil shear strength is hence decreased. If the pore pressure reaches the initial normal effective stress, there is no contact force remaining between sand grains as shown in Figure 2.1 (c). Loose soils can easily be disrupted by existing static forces, self weight, or external forces, such as earthquakes. Liquefaction can occur in both static and dynamic scenarios. Static liquefaction generally occurs after heavy rainfalls in loosely-deposited soil slopes. Due to the loose configuration of the soil skeleton, the loss of soil suction during rainfall infiltration can collapse the skeleton and generate shear induced pore pressures. In the dynamic condition, cyclic movement or an impulse force causes the soil particles to rearrange. If the soil was initially in a loose state, densification of the soil induces an increase in the pore pressure. A large displacement usually develops in this situation. If the soil is initially in a dense state, brittle failure and ground softening are expected.

Flow liquefaction occurs when the soil structure collapses because the residual strength of the soil is lower than the stress imposed by self weight. If the residual strength is greater than the static shear stress, a run-away failure will not occur, but the soil may experience significant stresses. If
this happens in saturated, gently sloping ground, it is called lateral spreading, as will be discussed later in Section 2.2.

2.1.2 Assessment of liquefaction
Liquefaction is usually found in shallow, loose, saturated, cohesionless soil deposits subjected to strong ground motions. Since loose soils contract when experiencing shear stress and the pore pressure builds up, they are more likely to be prone to liquefaction. Although it is recognized that the saturated sandy soils are prone to liquefaction, it is also possible for silts and silty sands to liquefy. Various researchers have proposed the method to assess the possibility of liquefaction.

Tsuchida (1970) proposed a grain-size distribution curve criterion to identify liquefiable soils as shown in Figure 2.2. Any sand that falls within the proposed boundary curves is prone to liquefaction. The liquefaction susceptibility of a soil was also empirically correlated to parameters such as intensity of ground shaking, depth of water table and the SPT blow-count of the soils. The correlations are mainly from Seed and Idriss (1982). An energy approach has also been used in calculation liquefaction assessment (e.g. Nemat-Nasser and Shokooh, 1979; Berrill and Davis, 1985; Figueroa et al., 1994; Thevanayagem et al., 2003). These energy approaches are based on the concept of the frictional energy loss required to liquefy the soils which is a function of density, confining stress, and frictional characteristics of the soil. The accumulation of energy loss up to liquefaction is used as an index indicating whether or not the liquefaction will occur.

2.2 Lateral spreading

2.2.1 The concept of lateral spreading
Soil shear strength in a loosely-deposited and gently-sloping ground is lowered during an earthquake due to increased pore water pressures. Although the mobilized soil strength remains higher than the strength required to resist the static shear forces, the combination of the inertia
forces from an earthquake and static shear forces can cause a large deformation in the mobilized soil. Because of the gentle slope, the liquefied soil is likely to move preferentially in one direction. Thus, the seismic induced downslope displacement in saturated sloping ground is usually larger than either unsaturated sloping ground or level ground. In this thesis, the term “lateral spreading” is used to refer to the downslope movement of the saturated, loosely-deposited, gently sloping ground during earthquake shaking.

Example of the consequences of lateral spreading from earthquakes in Peru and Guatemala are shown in Figure 2.3. During the 1970 Peru earthquake, lateral spreading caused damage to the paved road near the Bay Shore in western Chimbote. A few years later, the 1976 Guatemala Earthquake induced lateral spreading lead to a collapse of the slopping ground along the Motagua River.

**2.2.2 Analytical prediction of lateral spreading**

Newmark (1965) proposed a method to calculate slope movement in an earthquake. The concept is based on a sliding block on a frictional plane and the permanent movement is a result of the earthquake acceleration which exceeds the yield acceleration. Yield acceleration is the minimum acceleration required to initiate sliding of the block from the frictional surface. Since predicting the acceleration time history is difficult, different researchers suggested various approaches to determine the parameters for earthquake acceleration. Bazair (1991) and Bazair et al. (1992) proposed an equation based on the block sliding method and assumed the equivalent sinusoidal base acceleration to be the acceleration used in Newmark’s analysis. In order for this equation to be used in prediction of lateral spreading displacement during liquefaction, yield acceleration should be calculated in terms of total stress and undrained strength. To simplify the model, soil parameters are treated as constants throughout the liquefaction analysis and changes in soil behaviour due to liquefaction are neglected.
Based on Newmark’s model, an analytical method that considers the conservation of energy rather than the equilibrium of forces during slope deformation was proposed by Byrne (1991) and Byrne et al. (1992). However, in this model, knowledge of the nonlinear stress-strain relationship is required which adds complexity to the analysis. To comprise the variation of soil behaviour in Newmark’s model, Haigh (2002) proposed the Effective Stress Newmark Method with change in soil strength due to rising of pore pressure is considered. The excess pore pressure time history and the liquefied strength ratio are, therefore, required in the yield acceleration calculation.

An alternative analytical approach presented by Towhata et al. (1992) involves the development of a minimum potential energy model from two simple models. The first model is an inclined, linear-elastic, surface soil column subjected to axial compression. The second model is a flow model based on the energy required to change a liquefied sloping ground to level ground.

Both Newmark’s and Towhata’s models are fairly simple but contain restrictions and limits when used. It is difficult to define appropriate shear strengths for liquefiable soils and a yield acceleration as used in Newmark’s model. In addition, Newmark’s model is incapable of modeling lateral spreading deformation after earthquake motions stop. In contrast, Towhata’s model was developed from shake table model experiments which may not accurately represent field behaviour. The elastic stiffness of an unliquefied surface soil layer used in the method is poorly defined. The Towhata’s method only provides the maximum displacement and is limited to only simple geometry.

### 2.2.3 Empirical prediction of lateral spreading

Although the understanding of lateral spreading has increased, its analysis is complicated and time consuming. An alternative approach is to use empirical methods from data recorded from real earthquakes to attempt to predict the horizontal displacement of laterally spreading ground.
Hamada (1986) proposed an equation for the prediction of lateral spreading based on 60 cases of lateral spreading in Niigata (1964) and Nihonkai Chonbu (1983) in Japan and San Fernando Valley in California. Using this data base, Hamada (1986) estimated displacement according to the topography of the sites where soil liquefaction occurred. Depth of liquefiable layer and slope angle are used in this prediction. With this empirical equation, prediction is biased toward the site conditions used to generate the empirical relationship. A reduction of accuracy for the displacement prediction for slopes with dissimilar site conditions to Noshiro, Japan, should therefore be expected.

The liquefaction severity index was proposed by Youd and Perkins (1987) to predict the maximum displacement expected during lateral spreading. It is a function of the earthquake motion and the distance from the earthquake epicentre. The derivation of the calculation is based on the data from the western United States and Alaska earthquake case histories. The liquefaction severity index determined from this equation can then use to calculate the expected maximum ground failure displacement. The method is simple and research shows that it provides a conservative value of displacement.

Bartlett and Youd (1992, 1995) create a composite empirical relationship which includes the effect of the earthquake motion and the site conditions. A data base of 467 horizontal displacements from Japan and western United States were analyzed with a multi linear regression model (MRL) to create an empirical relationship for seismic-induced horizontal displacements. Two equations were proposed, one for a slope containing a vertical free face at its toe.

Youd et al. (2002) proposed new equations to supersede the previous model by Bartlett and Youd (1995). Additional case history data from the 1983 Borah Peak earthquake in Idaho, the 1989 Loma Prieta earthquake in California, and the 1995 Kobe earthquake in Japan were added. The
error in the data and boundary in the old model were corrected and the new equation was subsequently proposed.

Whereas Youd and his colleagues used the MLR model to quantify the lateral spreading displacement, Wang and Rahman (1999) introduced an artificial neural network model (ANN), "computational mechanism able to acquire, represent, and compute a mapping from a multivariate space of information to another, given a set of data representing the mapping", to lateral spreading studies. Other groups of researchers have employed similar computational techniques in lateral spreading studies (e.g. Baziar and Nilipour, 2003; Baziar and Ghorbani, 2005; Baziar and Jafarian, 2007; and Garcia et al., 2008).

2.2.4 Experimental studies

Centrifuge modeling has been used to model the phenomenon of lateral spreading to study this aspect of soil behaviour and the effects of geometrical and constitutive properties. Pilgrim (1993) observed steep and gentle liquefiable sand slope performance during earthquakes and suggested that, in small earthquakes, smaller displacements occur in steeper slopes because of the requirement for higher energy content to build up the excess pore pressure. Fiegel and Kutter (1994a; 1994b) conducted several centrifuge models to study the effect of layers on lateral spreading. The study showed that soil displacements were distributed over the height of the uniform sand layer and the large displacement was concentrated at the interface between the sand layer and the overlying silt layers which is relatively impermeable. The silt layers also restricted the pore water pressure dissipation while sand boils were observed at the area with the thinnest layer of less permeable layer. Taboada (1995) studied the effect of slope geometry and earthquake characteristics on slope performance and found that the progressive shearing due to liquefaction in slopes results in soil dilation resulting in the attenuation of excess pore pressure. Thus, the displacement of a lateral spreading slope is limited by the temporary increase of soil
shear strength. Haigh et al. (2000) used coloured sand marker lines to measure the displacement profile through the laterally spreading ground. Their results indicated that soil surface both displaced laterally and experienced rotation causing settlement at upslope and heave at downslope portion of the model, Figure 2.4.

2.3 Soil-pile interaction

2.3.1 General background

Piles are generally used as deep foundations to transfer loads from superstructures to competent strata. Load transfer from piles to soils consists of two mechanisms: bearing forces at pile tips, and frictional forces acting on the surface area of the pile. However, piles sometimes undergo lateral loads (e.g. wind loads or earthquakes) and the soils therefore resist horizontal pile movement. For piles embedded in loosely-deposited sands subjected to earthquakes, moving soils will impose lateral drag forces on piles. Soil and pile responses are unable to be calculated explicitly. An understanding of the interaction between the soil and piles is hence necessary for pile design.

2.3.2 Dynamic response analysis methods for piles

Several researchers have developed methods to analyze pile response to seismic events using finite element methods (e.g. Angelids and Roesset, 1980; Randolph, 1981; Faraque and Desai, 1982) and boundary element methods (e.g. Sanchez, 1982, and Sen et al., 1985), with soil being treated as a continuum. Due to its complexity, dynamic numerical modeling is currently being used only in large projects. Instead, the simplicity of the Beam on Nonlinear Winkler Foundation (BNWF) model which uses a nonlinear soil pile (p-y) spring as the soil-pile interaction is commonly used. In the BNWF analysis, a series of beam-column elements represent soils which each have discrete springs connecting them to the piles. Matlock et al. (1978) developed a program based on BNWF concept called SPASM (“Seismic Pile Analysis with Support Motion”)

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for analyzing dynamic soil-pile interaction. In this model, p-y springs were excited by the ground motion time history along the depth of the soil profile. Kagawa (1980) suggested the addition of a dashpot to the p-y spring model can be used to introduce the effect of damping. Wang et al. (1998) subsequently substituted the pile displacement by the relative displacement between far-field and pile displacement for a more accurate response.

2.4 Pile in laterally spreading ground

2.4.1 General background
Lateral spreading has the potential to cause catastrophic damage to infrastructure. The damage of piles supporting bridges is one of the most typical failures that occur during lateral spreading. The design of piles in lateral spreading soils was first issued after the bridge collapses in the 1964 Alaskan Earthquake and the 1964 Niigata Earthquake. Figure 2.5 illustrates the collapse of Showa Bridge in Niigata. In this case, the deformation of a pile caused the pier head to deflect by 1 m leading to the bridge span falling off from the bridge support.

2.4.2 Design codes for piles in lateral spreading soil
Japanese Highway Bridge Specification (JRA 1996) or (JRA 2002)
In response to the 1964 Niigata Earthquake bridge failure, Japan Road Association, JRA, proposed a design concerning piles subjected to lateral spreading soils called “Seismic coefficient method” (Japanese Highway Bridge Specification, JRA 1972). Japanese engineers subsequently found that the JRA (1972) design recommendation over estimated the forces imposed on a pile during liquefaction which led to the introduction of the new Japanese Highway Bridge Specification in 1980 (JRA 1980). Pile foundations were designed based on this version of the code until the massive destruction of bridges in the 1995 Kobe Earthquake. The JRA 1980 was consequently substituted by the Japan Highway Bridge Specification (JRA1996).
The design recommendations state:

“In a case where the effects of lateral spreading are accounted, the effect of lateral spreading shall be provided as horizontal force to study the seismic performance of the foundation. But in this case, it shall not be necessary to simultaneously account for the inertia force produced by the weight of the structure.”

“...If laterally spreading ground does not contain a non-liquefied crust, the horizontal force will be calculated from 30% of the total overburden pressure.”

(JRA 1996)

There were many experiments carried out to investigate the performance of the JRA specifications. Sato et al. (2001) studied the accuracy of the JRA (1996) by using dynamic centrifuge modeling. They found that JRA (1996) overpredicted the pressure imposed on a pile foundation located behind a retaining wall both during and after earthquakes. Centrifuge tests done by Dobry and Abdoun (2001) shows that the magnitude of pressure from the experiments agrees well with JRA (1996) prediction. In contrast, the dynamic centrifuge modeling study of Haigh and Madabhushi (2002) found that JRA (1996) underestimated the transient lateral earth pressures during shaking but gave reasonable predictions at post-earthquake.


The National Earthquake Hazard Reduction Program (NEHRP) briefly mentions about the design for piles responding to earthquake in the NEHRP (2000):

“If an unloaded pile were placed in the soil, it would be forced to bend similar to a pile supporting a building. The primary requirement is stability, and this is best provided by piles that can support their loads while still conforming to the ground motions and, hence the need for ductility”.

P-Y Curve (API 1993)
The American Petroleum Institute standard (API) has recommended that pile design for lateral loading be performed using parallel nonlinear soil-pile springs (p-y). The design is based on the results of static and cyclic lateral load tests and the mechanism of seismic soil behaviour which leads to lateral spreading (i.e. pore pressure generation) is neglected leading to greatly simplified liquefaction pile design. Liu and Dobry (1995) performed centrifuge tests to investigate pile responses in liquefied sands and derived scaling factors for the p-y spring for piles during liquefaction.

2.4.3 Experimental studies
At Rensselaer Polytechnic Institute (RPI), Abdoun et al. (2003) and Dobry et al. (2003) studied eight centrifuge models of single piles and pile groups subjected to earthquake-induced liquefaction and lateral spreading. Model piles of 50 cm diameter were installed in a mildly sloping, submerged ground profile containing a liquefiable Nevada sand layer. Abdoun et al. (2003) focused on comparing the pile bending moments of end-bearing and floating piles, single piles and pile groups, piles with and without a pile cap, and piles in two and three layer system. Results show that the maximum bending moment occurred at the boundary between liquefied and non-liquefied layers. They found that the bending moment reached a maximum during the shaking and then decreased toward the end of the earthquake. Dobry et al. (2003) used the centrifuge results to calibrate the two simplified limit equilibrium (LE) methods for estimating the bending moment response and therefore a factor of safety against bending failure of the pile during lateral spreading. The two proposed methods were verified using the data of the Niigata Family Court House (NFCH) building in the 1964 earthquake.

Meanwhile at Cambridge University, Haigh (2002) and Haigh and Madabushi (2002) carried out a series of centrifuge tests which concentrated on the variation of pore water pressures close to the pile and the resultant bending moments of the pile in liquefied soil. The model had a pre-
installed pile that was embedded in a submerged 6 degree slope of liquefiable sand. These researchers found that cyclic pore pressures observed around the pile were much higher than in a free-field case. Furthermore, pore pressure cycles observed at the downslope of the pile were higher than observed upslope of the pile. As mentioned in the JRA (1996) code, the bending moments in the centrifuge tests were compared with the bending moments calculated from JRA (1996) – the results show that JRA (1996) under predicts pile bending moments during the transient loading. Using this observation, Haigh and Madabushi (2002) argued that the bending moment should be a function of liquefied sands displacement rather than the earth pressure as used in JRA (1996). In addition, Haigh and Madabushi (2006) drew stress paths from the data of pore pressure transducer and stress cells which were measured at the front and back of a pile against lateral spreading in centrifuge tests. They demonstrated that the stress paths follow the passive failure line at the upslope of the pile, whereas paths cycle between active and passive failure lines at the downslope of the pile. When comparing square piles and circular piles, the observations showed that the residual bending moment in square piles is 8% higher than in circular piles, whilst the bending moment in square piles was more than 30% higher than in circular piles during shaking. Moreover, large suction spikes occur during early cycles, while conciliation of maximum horizontal total stress and minimum pore pressures occurred later in the earthquake – the result is a significant effect on the shape of the stress paths.

At the University of California in San Diego, Ashford et al. (2002) carried out full scale experiments on Treasure Island in the San Francisco Bay to observe pile performance and soil response under lateral loading during liquefaction in level ground. The study was focused on pore water pressure responses observed by piezometers adjacent to piles. Phase transformations of pore water pressures were observed in soils around the pile and adjacent soils at a distance of 4.2 m from the pile. The phase transformation was explained as a significant factor in soil providing resistance to laterally loaded piles. Additionally, two-full scale experiments were conducted on
the Port of Tokachi on Hokkaido Island, Ashford et al. (2006) and Juinrarongrit and Ashford (2006). The study used blasting induced laterally spreading ground to observe the behaviour of single piles and pile groups subjected to lateral spreading. Lateral forces imparted on piles were back calculated from bending moment data measured by strain gages. The experimental results were subsequently compared with a pseudo-static push over analysis using the p-y analysis method. The soil-pile interaction was modeled either as a zero stiffness spring or using p-y curves from a p multiplier approach. The zero soil spring stiffness approach showed good agreement with experimental results while the p multiplier approach tended to overestimate the response of the pile to lateral spreading.

2.5 Scaling law for dynamic centrifuge

Geotechnical centrifuge modeling is a powerful and extremely useful experimental technique in which complex geotechnical phenomena can be studied using small scale models. The idea of a centrifuge is to reduce the size of the model and maintain the prototype effective stress by increasing the g-field. A scaling factor is required to translate the results to full scale. A full derivation of scaling laws can be found in ISSMGE TC2 (2007) and a derivation for this technique is reported by Schofield (1981) and Kutter (1994) and is summarized in Table 2.1.
Table 2.1 Scaling factor for dynamic centrifuge

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Scaling law model/Prototype</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>General scaling laws (non-dynamic events)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>1/n</td>
<td>m</td>
</tr>
<tr>
<td>Area</td>
<td>1/n²</td>
<td>m²</td>
</tr>
<tr>
<td>Volume</td>
<td>1/n³</td>
<td>m³</td>
</tr>
<tr>
<td>Mass</td>
<td>1/n³</td>
<td>Nm⁻¹s⁻²</td>
</tr>
<tr>
<td>Stress</td>
<td>1</td>
<td>Nm⁻²</td>
</tr>
<tr>
<td>Strain</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>Force</td>
<td>1/n³</td>
<td>N</td>
</tr>
<tr>
<td>Bending moment</td>
<td>1/n³</td>
<td>Nm</td>
</tr>
<tr>
<td>Work</td>
<td>1/n³</td>
<td>Nm</td>
</tr>
<tr>
<td>Energy</td>
<td>1/n³</td>
<td>J</td>
</tr>
<tr>
<td>Seepage velocity</td>
<td>1/n</td>
<td>ms⁻¹</td>
</tr>
<tr>
<td>Time (consolidation)</td>
<td>1/n²</td>
<td>s</td>
</tr>
<tr>
<td>Dynamic events</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Time (dynamic)</td>
<td>1/n</td>
<td>s</td>
</tr>
<tr>
<td>Time (Seepage)</td>
<td>1/n²</td>
<td>s</td>
</tr>
<tr>
<td>Frequency</td>
<td>n</td>
<td>s⁻¹</td>
</tr>
<tr>
<td>Displacement</td>
<td>1/n</td>
<td>m</td>
</tr>
<tr>
<td>Velocity</td>
<td>1</td>
<td>ms⁻¹</td>
</tr>
<tr>
<td>Acceleration/acceleration due to gravity (g’s)</td>
<td>n</td>
<td>ms⁻²</td>
</tr>
</tbody>
</table>

2.6 Conclusions

A significant amount of research has been performed to investigate soil behaviour during lateral spreading and to develop methods of analysis for piles in laterally spreading slopes. However, limitations in current technology not yet allowed the measurement of full-depth in-cycle soil displacements during lateral spreading. In this study, a high-speed camera, coupled with digital image analysis techniques, will be used to generate this data with an aim to improving our understanding of the mechanism of lateral spreading in liquefied soils.

The P-y curve approach has been widely used to model pile-soil interaction in laterally loaded piles because of its simplicity. However, a consequence of the lack of full-depth in-cycle displacement data is a certain level of uncertainty as to the shape of the P-y relationship for piles
in lateral spreading soil. This study aims to address this limitation and compare the observed P-y relationships to existing methods.
Figure 2.1 Illustration of soil deposits a) before liquefaction, b) when pore water pressure is increasing and c) during liquefaction (blue bars and arrows represent pore water pressure and contact forces between grains, respectively)
Figure 2.2 Boundary curves for liquefiable soils (Tsuchida, 1970)
Figure 2.3 Lateral spreading in a) the 1970 Peru Earthquake and b) the 1976 Guatemala Earthquake (U.S. Geological survey)
Figure 2.4 Lateral displacement and rotation of soil surface (Haigh et al., 2000)

Figure 2.5 Bridge collapse due to lateral spreading in the 1964 Niigata Earthquake
(Haigh, 2002)
Chapter 3
Quantification of Transient Soil Deformation Due to Lateral Spreading

3.1 Introduction

Lateral spreading is the downslope inertial mass movement of gently-sloping deposits of liquefiable sands during earthquakes. It has resulted in severe socio-economic damage to a wide variety of structures and properties. One of the most widely referenced examples of lateral spreading is the earthquake triggered collapse of the Showa River Bridge during the 1964 Niigata earthquake in Japan. The laterally spreading soil caused the piles supporting the bridge to deflect, increasing the distance between piers, and hence allowing the decks to fall from the piers. During the Alaskan (1964), Loma Prieta (1989) and Kobe (1995) earthquakes, lateral spreading also played a major role in the damage to civil engineering structures.

Although awareness of the devastation that can be caused by earthquake induced lateral spreading is rising, the complexity of the soil mechanics involved has limited the understanding of the mechanisms occurring and our ability to predict the displacements arising from this phenomenon. The first attempt was by Newmark (1965), who simplified the slope movement in earthquake to a sliding block. It has subsequently been modified to estimate the displacement of the sloping sand deposit during the earthquake by means of replacing the drained shear strength with the normalized undrained shear strength of liquefied soil in the yield acceleration term by Bazair (1992). Another analytical method involves the consideration of conservation of energy rather than equilibrium of forces during slope deformation (Byrne 1992). An alternative analytical approach (Towhata, 1992) involved the development of a minimum potential energy model from two simple models, one being an inclined linear elastic soil column resisting axial compression and the other a flow model in which flow energy is calculated based on the surface level.
Hamada (1986) carried out empirical prediction of displacement during lateral spreading by making the observed displacement a function of the thickness of the liquefiable layer and the slope gradient. The empirical analysis is based on 60 cases of lateral spreading in the Niigata (1964) and Nihonkai Chonbu (1983) earthquakes in Japan and the Fernando Valley (1971) earthquake in California. A year later, Youd and Perkins (1987) used the liquefaction severity index calculated from earthquake magnitude and the distance from earthquake epicentre as an index number to predict the expected maximum ground failure displacement. Bartlett and Youd (1992, 1995) introduced a new empirical equation which associated displacement with the effects of earthquake ground motion and site conditions using multi linear regression. Youd and Hansen (2002) updated the empirical relationship by adding more earthquake data and by correcting incorrect calculations and estimation in the previous model. Other approaches (i.e. artificial neural network and genetic algorithm) have been applied to the lateral spreading prediction by many researchers.

Experimental data on displacement that could be expected in laterally spreading ground has primarily taken the form of geotechnical centrifuge modeling study. Pilgrim (1993) observed steep and gentle liquefiable sand slope performance during earthquakes and concluded that, in small earthquakes, smaller displacement occurred in steeper slopes because of the requirement for higher energy content to build up the excess pore pressure. Fiegel and Kutter (1994a, 1994b) conducted several centrifuge model tests to study the effect of layers on lateral spreading. The study showed that soil displacements were uniformly distributed over the height of the sand layer in the case of homogeneous sand layers. The displacement was concentrated at the interface between the sand layer and the overlying silt layers in the case of a layered sand and silt deposit layers. The relatively low permeability of the silt layers also restricted the pore water pressure dissipation and sand boils were only observed in areas in which these layers were thinnest. Taboada (1995) studied the effect of slope geometry and earthquake characteristics on slope
performance and mentioned that the progressive shearing due to liquefaction in slopes allows soil
dilation resulting in attenuation of excess pore pressure. Thus, the displacement is limited by the
increase of soil shear strength.

It is important to note that only total post-earthquake slope deformation and surface displacement
have been measured in earlier studies (e.g. Haigh (2000) used coloured sand marker lines to
measure the displacement through the slope profile at the end of the test). The displacement
behaviour during an earthquake has either been neglected or inferred through the measurement of
accelerations, as there has been no technology to look at displacement behaviour during each
cycle of the earthquake. Moreover, previous research seldom addresses how soil deformation
responds to acceleration and the effect of negative excess pore pressure. This chapter reports the
results from a series of dynamic centrifuge model tests which were performed to make use of the
advances in digital image analysis and high speed photography to measure the transient
deformation response of soil in a laterally spreading slope. This chapter discusses the transient
physical behaviour of the laterally spreading slope, including acceleration, excess pore pressure
and displacement during the earthquake motion.

3.2 Methodology

3.2.1 Facility

Centrifuge modeling has been widely used in the study of the behaviour of lateral spreading
slopes due to its advantages in providing a cost-effective means of achieving realistic soil
behaviour in reduced scale models. Experiments were carried out using the C-CORE
Geotechnical Beam Centrifuge in St John’s, Newfoundland, Canada. The 5.5 m working radius
centrifuge is a 200 g-ton machine, with a payload capacity of 0.65 tons at 200 g and 2.2 tons at
100 g. The testing platform has physical dimensions of 1.1 m by 1.4 m in plan and 1.2 m in
height and is equipped with a Servo-hydraulic earthquake actuator manufactured by Actidyn
which uses servo-control to deliver the desired motion to the model (Phillips, 2002). A model container with two transparent acrylic sides, (756 mm x 286 mm in plan and 570 mm in height) was fabricated for use with this project to enable visual observation of the lateral spreading slope in plane strain. However, the full transparent face of the box cannot be used for optical measurements since the components of the earthquake actuator obstruct the view. To partially overcome this issue, one accumulator was removed from the actuator to increase the available field of view, however despite this, the field of view for camera is constrained to the left corner of the front view of the box. Figure 3.1 shows the box dimension and the field of view.

### 3.2.2 Test configuration

Two centrifuge models were tested, each consisting of 6 degree slope comprising a 200 mm thick layer of saturated loose sand, ID ~40%, underlain by dense sand, ID ~ 80%. Each model has a highly instrumented pile embedded through the loose sand layer and 60 mm into the dense sand layer. A circular model pile was placed at 200 mm from the left end of the box and on the centerline between front and back of the model pile. Two types of earthquake motion were applied to each model. To simplify the complexity of real earthquake motion, 50 cycles of a 50 Hz 10 g sinusoidal wave was used as the first input motion for each model. A second event was then modeled using the COSTA-A input motion which is a simulation of the 1994 British Columbia Earthquake, Locat et al. (2001). The model geometry and test details are shown in Figure 3.2 and Table 3.1.
Table 3.1 Test detail

<table>
<thead>
<tr>
<th>Test model</th>
<th>Test</th>
<th>Input motion</th>
<th>Pile location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>RK1</td>
<td>Sinusoidal (50Hz)</td>
<td>Center</td>
</tr>
<tr>
<td></td>
<td>RK2</td>
<td>COSTA-A</td>
<td></td>
</tr>
<tr>
<td>Model 2</td>
<td>RK3</td>
<td>Sinusoidal (50Hz)</td>
<td>Center</td>
</tr>
<tr>
<td></td>
<td>RK4</td>
<td>COSTA-A</td>
<td></td>
</tr>
</tbody>
</table>

3.2.3 Instrumentation

In order to study the behaviour of the soil in the slope during shaking, a number of high performance instruments were used during the test. As shown in Figure 3.2, four ICP miniature accelerometers (ACCs) were located at mid-depth and full-depth of the loose sand layer to observe the acceleration response of the slope. To measure the excess pore water pressure development during the tests GE-DRUCK PDCR81 miniature pressure transducers (PPTs) were placed at four different positions in the slope profile during sample preparation. Ten sets of full bridge strain gages (manufacturer type EA-06-060PB-350) were attached on the model pile to monitor pile bending moment and two linear variable differential transformers (LVDTs) were positioned horizontally at the top of the model pile to measure pile head displacement and rotation. A Phantom V9.0 high speed camera was used for investigating displacement over the cross section of the slope. This camera is capable of a frame rate of 1000 fps at a resolution of 1632x1200 pixels. A high intensity DC light source was selected in order to overcome the short exposure time to be used by the camera and to have the required durability to work in the 50 g gravity-field. With data recorded at 1 kHz for ACCs, 2 kHz for PPTs and strain gages and 1 kHz for images, the mechanical and physical behaviour of the soil can be investigated in great detail during shaking. Since the behaviour of laterally spreading ground is the focus of this chapter, discussions on the results of transient pile deformation will be discussed for chapter 4.
### 3.2.4 Material properties

Fraser River Delta Sand is a uniform fine sand comprised of very young sediments that have been rapidly deposited at the mouth of the Fraser River in British Columbia, Canada. Thus, it is characteristic of materials that are susceptible to flow liquefaction and flow slides. This material has also been extensively used in liquefaction studies conducted by C-CORE and the University of British Columbia (UBC), Canada. Wijewickreme et al. (2005) determined the dynamic parameters of this soil by cyclic direct simple shear tests on specimens prepared using the air pluviation technique, which are included in Table 3.2.

**Table 3.2 Properties of Fraser River sand (Wijewickreme et al., 2005)**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum void ratio ($e_{\text{max}}$)</td>
<td>0.93</td>
</tr>
<tr>
<td>Minimum void ratio ($e_{\text{min}}$)</td>
<td>0.62</td>
</tr>
<tr>
<td>Mean diameter ($D_{50}$)</td>
<td>0.26 mm</td>
</tr>
<tr>
<td>Specific gravity ($G_s$)</td>
<td>1.6</td>
</tr>
<tr>
<td>Uniformity coefficient ($c_u$)</td>
<td>2.71</td>
</tr>
<tr>
<td>Cyclic resistance ratio (CRR)</td>
<td>0.08 (Dr = 40%)</td>
</tr>
<tr>
<td></td>
<td>0.35 (Dr = 80%)</td>
</tr>
<tr>
<td>$K_\alpha$</td>
<td>~0.8</td>
</tr>
</tbody>
</table>

1 Cyclic resistance ratio measured on sample with vertical effective stress of 100 kPa and number of cycles to liquefaction of 10 cycles with the condition of initial static shear stress is zero
2 Factor from normalized initial static shear stress level on liquefaction resistance

### 3.2.5 Preparation technique

The first stage of constructing the centrifuge model was to place gravel or coarse sand at the base of the box to create a highly permeable layer at the bottom boundary to ensure uniform saturation. Two pieces of Duxseal (a sealing and caulking compound) were then attached at the left and right sides of the box as a damper to reduce the effects of the rigid sidewalls on the dynamic behaviour
of the model (see Teymur 2002 for a detailed discussion of boundary effects in dynamic centrifuge models). In this case, a use of Duxseals is based on the previous study of Chakrabortty (n/a). To achieve the desired soil density, the sample preparation technique by means of the dry air pluviation was employed. The orifice size in the sand hopper, hopper height and the rate of hopper movement were calibrated before use and a combination was chosen to achieve the desired sand density. During the model making process, the air pluviation process was periodically paused and the highly-instrumented pile and instruments including PPTs and ACCs were placed at the appropriate elevation in the model with care. The ACCs were placed in the same direction as the slope inclination and the input motion. Initial void ratios for the loose and dense layers are 0.812 and 0.684, respectively. This corresponds to an initial density of 1.496 g/cm$^3$ for the loose layer and 1.609 g/cm$^3$ for the dense layer.

Once the soil model was completed, it was saturated with a viscous pore fluid. The viscous fluid was chose to correct the different in time scale of dynamic time and seepage and dissipation time in centrifuge scale. Methocel F 50 powder was mixed with water to generate a pore fluid with a viscosity of 50 cSt. Because of the high viscosity of the fluid and the need to achieve full saturation, a vacuum was applied to the model to remove the air trapped in the voids. Carbon dioxide gas was thus introduced to displace any remaining pore air before the viscous pore fluid was allowed to slowly saturate the model from the bottom up. Full details of the viscous fluid preparation and the saturation process are reported by Pfister (2006).

3.3 Results

3.3.1 Models
Two models were tested and two earthquake motions were applied to each model. Each of the two lateral spreading models is expected to produce similar behaviour as they are similar soil models subjected to similar earthquake events. In order to validate the assertion that the models
have nominally similar soil profiles, a cone penetration test using a miniature cone for a centrifuge test was performed prior to each earthquake. These results are shown in Appendix A and confirm this assertion.

3.3.2 Slope acceleration

Soil acceleration and pore pressure generation were observed during the test by means of ACCs and PPTs. The acceleration was applied at the base of the box and transmitted vertically to the soil layer by propagation of shear stresses. The acceleration measured in the soil layer varied through the depth due to the shear stiffness which was altered by the seismic event. Figure 3.3 shows the base acceleration and acceleration response of accelerometers ACC1 and ACC3 which were located at the middle and base of the loose sand layer, respectively. The data is plotted in both the time domain and the frequency domain by using the Fast Fourier Transform (FFT) to help the better observation of acceleration attenuation. It also needs to be noted that the positive acceleration in the graph corresponds to the downslope direction whilst the negative value corresponds to the upslope direction.

An input motion consisting of 50 sinusoidal cycles with amplitude 10 g and frequency 50 Hz was applied at the base of the box as shown in Figure 3.3 (c). The maximum Fourier amplitude is about 23000 g/Hz at 50 Hz without any significant response at lower frequencies. Although there are several small size amplitudes at higher frequencies, these have minor energy content when converted into the acceleration time history. A similar response is shown in ACC1 which is at the base of the loose sand layer, shown in Figure 3.3 (b), except that a few higher frequency components appear in ACC1. At mid-depth in the loose sand layer the soil response shows the shrinking of the Fourier amplitude at 50 Hz from 23000 g/Hz in ACC1 to 17000 g/Hz in ACC3, as shown in Figure 3.3 (a). Moreover, an additional peak height of 830 g/Hz appears at 25 Hz. The acceleration response at the mid-depth of liquefiable layer is significantly different from the
input acceleration. In particular, the acceleration time history shows attenuation in the up-slope direction while both attenuation and large spikes in the down-slope direction. The trend of large spikes at alternate peaks increases until it reaches peak value at 1.4 s and then slightly decreases.

The Fast Fourier Transform is a useful tool for dynamic analysis, but it also has some disadvantages. Because an FFT is calculated over the full length of the signal, its outcome shows an average of the frequency content over the entire time-span of the signal. Therefore, an FFT cannot represent what actions happen during a short period in the time series and cannot locate when these actions take place. This is important as the acceleration time history of the liquefied layer (Figure 3.3a) clearly shows that the frequency content of the soil acceleration changes with time in the liquefied zone.

To investigate the temporal variation of the soil response in term of acceleration, wavelet analysis has been performed following the procedure of Newland (1997), Haigh (2002) and Haigh et al.(2002). In brief, a wavelet is a mathematical tool which decomposes the signal into components called levels and then converts each level into functions of basic shape. Thus, it is capable of displaying data in the time-frequency domain unlike traditional FFT.

The wavelet analysis results (often referred to as wavelet maps) are shown in Figure 3.4 where the dark color represents lower level of amplitude while the light color represents a higher level of amplitude. The map of ACC3 in Figure 3.4 (b) varies from the base acceleration Figure 3.4 (c) only in that some high frequency components are perceived which is similar to what we have seen in the FFT analysis. In Figure 3.4 (a), the amplitude at frequency 50 Hz becomes smaller after 0.9 second and the amplitude at 25 Hz increases slightly at 0.8 second, reaches the largest value at 1.4 second and is stable until the applied motion stops.
3.3.3 Pore pressure response

Excess pore pressure (EPP) data from pore pressure transducer readings is very useful for indicating if and when liquefaction takes place. When the excess pore pressure increases to the initial vertical effective stress, effective stress is essentially zero and the soil has no strength to resist shear stresses. The pore pressure transducers were installed at a depth of 34 mm and 134 mm under the soil surface. For full liquefaction, therefore EPP reading of 16 kPa and 58 kPa are expected at these two locations, respectively. From Figure 3.5 (a) EPP reading at the surface, reach the effective stress corresponding to full liquefaction at an elapsed time of 0.9 seconds and slightly increase until the earthquake motion stops. At mid-depth, liquefaction is achieved slightly later at 1.1 seconds as shown in Figure 3.5 (b). Interestingly, negative EPP spikes are observed in every alternate cycle at the surface, dropping in magnitude from -20 kPa at the beginning of the earthquake to essentially 0 kPa pore pressure at the end of the shaking. Also of note is that negative excess pore pressures are significantly lowered in magnitude as observed in the mid-depth reading.

3.3.4 Lateral spreading displacement

Particle image velocimetry (PIV) is an image processing technique based on digital image correlation which can be used to measure the relative displacement between subsequent digital images. PIV has been used widely in many areas of experimental research and the implementation of PIV for geotechnical engineering (geoPIV) used in this study is described by White et al. (2003). This technique works by tracking patches from the first image, in this case, each containing approximately 32x32 pixels in the second image to sub-pixel accuracy. Once this procedure has been carried out for all patches in the initial image, the displacement field will have been measured. In the experiments described here, a Phantom v9, high speed digital camera was used to acquire the images which were then analyzed using geoPIV in order to measure the
evolution of soil displacement in laterally spreading slopes. The total displacement measured in the cross section of the slope at the end of the lateral spreading event is shown in Figure 3.6.

It can be difficult to use geoPIV to follow soil deformation during a liquefaction event as the liquefied soil shears and mixes (especially at the surface) resulting in changes in the visual texture of the patches during the shaking. It is hence important to acquire images at a rapid rate in order that this distortion is minimized between images to allow for accurate patch tracking. The use of geoPIV in situations involving liquefaction hence needs optimizing to obtain accurate geoPIV results. As can be seen from the results, no vectors are shown in the surface area as this was the most disturbed area due to soil shearing and mixing under the influence of the outflow of pore water from the underlying liquefied soil.

Figure 3.6 shows the total displacement measured post-earthquake through the slope profile. The top surface of the soil slope experienced horizontal displacements of around 30 mm corresponding to 1.5 m at prototype scale and reduced in angle from 6° to 3.5°. The PIV results show a shear zone that dips at an angle greater than the slope surface as a result of boundary effects. An angle of shear zone is expected to be more similar to a slope angle when a distance from the boundary is further. It can be seen that below a depth of 110 mm, the dense sand layer shows very little accumulated displacement during the earthquake, as would be expected owing to the non-liquefiable nature of the dense sand. Generation of excess pore pressures may have occurred even in this dense sand deposit (Coelho et al. 2007), although as no instrumentation was present at this location, this cannot be confirmed.

Figure 3.7 shows the evolution of surface displacement with time as the earthquake occurs. It can be seen from this figure that the displacement increases gradually throughout the earthquake showing a stepped response with steps being observed in alternate earthquake cycles. This will be discussed further in Section 3.3.5 with referenced to the negative pore-pressure spikes mentioned
in the previous section. The rate of generation of displacement is approximately constant through the earthquake and stops when earthquake shaking ceases as the residual strength of the liquefied soil is sufficient to maintain slope stability once the effect of the inertial loads is removed.

3.3.5 Co-seismic behaviour

High speed camera technology used together with PIV allows a unique insight into the evolution of soil displacement during the earthquake. In this section, the displacements of a soil column through the full depth of the liquefiable layer within 2 cycles of motion will be discussed. Two cycles of motion rather than one must be analyzed owing to the presence of suction spikes in alternate cycles of the earthquake as discussed in Section 3.3.3 and the steps in displacement exhibited in alternate cycles of the earthquake as discussed in Section 3.3.4.

Figure 3.8 shows the displacement of the soil column from 1.04 s to 1.08 s relative to its position at the beginning of this time interval. For the acceleration trace shown in Figure 3.8 (a), positive accelerations correspond to accelerations towards the toe of the slope. It can be seen that the slope movement is concentrated in the liquefiable layer above an elevation of 250mm, with no significant movement being seen below this level. It also can be seen that the movement occurs in the A-C and E-G time intervals (Figure 3.8 b and h) are less than in the C-E and G-I time intervals (Figure 3.8 b and h) which correspond to times when the base of the model was accelerating upslope hence applying the maximum shear stress to the soil as the inertial and gravity forces are both acting downslope. It is interesting to note that, although, a similar earthquake cycle was applied, the displacement in G-I time interval was measured much larger than the displacement in C-E time interval.

To observe shapes of the soil column displacement, the displaced shape of the soil column were normalized by the maximum displacement at each time step as shown in Figure 3.8 (c), (e), (g) and (i). It can be seen that the shape of the soil displacement profile varies slightly during the
shaking, but in general could be described by a linear function representing the average soil displacement superposed by a half sine wave which is positive during periods of downslope acceleration and negative during periods of upslope acceleration.

In order to understand the processes leading to this evolution of slope displacement, the path of a single patch of soil at mid-depth within the liquefiable layer displacement is plotted in Figure 3.9, together with box acceleration, soil acceleration, soil velocity and pore pressure. For the time period under consideration, the box acceleration and soil acceleration are shown in Figure 3.9 (b) in which box acceleration shown by the dark line and soil acceleration by the light line. The resulting soil velocity and excess pore pressure are shown in Figure 3.9 (c) and (d), respectively. The soil behaviour is split into 4 time intervals indicated in the figure as A, B, C and D in order to clarify discussion of the soil behaviour. In terms of input motion, interval A is similar to C and B is similar to D, but the excess pore pressures resulted in different soil response.

During interval A, the base acceleration is in an upslope direction, pore pressure increases by around 10 kPa and the soil moves downslope by 0.9 mm horizontally and by 0.1 mm vertically. While the pore pressure increases, the soil velocity increases to 9 cm/s (velocity can be calculated by differentiation of the positions measured by PIV). During interval B, the base accelerates downslope and the soil tends to flow back up the slope by half the distance of interval A or around 0.5 mm horizontally and 0.1 mm vertically. Excess pore pressure decreases by about 10 kPa and velocity stayed slightly lower than zero throughout the interval. During interval C, downslope velocity builds up to 14 cm/s at which point a rapid drop in pore-pressure is observed due to soil dilation. This also results in an extra spike in acceleration instead of the smooth decrease of acceleration observed in interval A as the soil is brought to a halt by the increase in shear strength that the soil exhibits in this lower pore-pressure and hence higher effective stress state. Once the soil has stopped moving, pore-pressure once again increases as the monotonic
shear strain increase which caused the dilation to now be stopped. During interval C the soil moves downslope by 1.5 mm horizontally and 0.6 mm vertically. During interval D very similar behaviour is seen to that in interval B with the soil sliding upslope by around 0.2 mm as pore-pressure gradually falls.

To confirm the double cyclic behaviour, displacement in profile slope was calculated using the PIV technique. Similar to single soil path and soil column displacement, the double-cyclic behaviour was also observed throughout profile slope as shown in Figure 3.10 (a) to (d). Liquefying layers moved downward when box accelerated upward, in interval A and C, but greater displacement was observed in interval C when a sudden drop of acceleration appeared in soil acceleration responses.

3.4 Discussion

The data was compared to the study in Cambridge, UK by Haigh (2002). The observed total displacement of 30 mm, corresponding to 1.5 m in real scale, is similar to previous study.

The double cycle nature of the displacement response of the lateral spreading slope is significant. It significantly alters the irrecoverable displacement of each cycle of the earthquake record which subsequently impacts total accumulated displacement of the slope. This impact on the free-field deflection is of significance as it will greatly influence the interaction between the liquefied soil and pile foundations.

The double cycle is a result of the occurrence of the 25Hz frequency component in resulting soil acceleration response. Although the amplitude of this peak in the FFT is not enormous, the small amplitude at low frequency has a huge effect on the acceleration characteristics which can be seen in the time history of acceleration. It is interesting that many high frequencies appeared since the earthquake motion was applied whereas the 25Hz frequency appeared later during the motion as seen in resulting wavelet maps.
Calculation of shear wave velocity is employed to determine the soil column natural frequency which is expected to be the source of double cycle behaviour. Another application of signal processing employed in this study is the cross correlation technique for the calculation of time lag between two adjacent acceleration readings. The calculated time lag is subsequently used to divide the distance between two accelerometers to determine the shear wave velocity. As shown in Figure 3.11, the shear wave velocities for the dense sand layer measured at both up-slope and down-slope sides are very scattered but show a reduction of velocity from 220 m/s to approx 100 m/s, whereas those for the loose sand layer measured at the up-slope side also show large scatter and a reduction in velocity from 220 m/s to 50 m/s. In the loose layer at the down-slope side, the shear wave velocity dropped from 225 m/s to 20 m/s at 0.9 second or within 15 cycles.

The time 0.9 sec when shear wave velocity dropped corresponds to the time when pore pressure reached the expected value to cause liquefaction and the time when the additional low frequency component observed in the resulting wavelet maps. Moreover, soil column natural frequency can be calculated from shear wave velocity divided by a wave length which, in case of producing resonance, is equal to column height timed four. The 25Hz soil column natural frequency determined from the shear wave velocity and slope height of 200 mm is compatible with the 25Hz frequency component observed from the soil acceleration response.

Since 25Hz is half of the input motion of 50Hz, the effect of input motion characteristic is a concern. The COSTA-A input motion simulating the 1994 British Columbia Earthquake was used as a second motion and then applied to the model following the sinusoidal motion. The fact that the first motion is higher energy content and denser packing leads to that the soils showed less response during second motion. In spite of less response, the appearance of the 25Hz frequency component is observed in the COSTA-A motion as shown in Appendix B, together with other responses. Therefore, the characteristic of the shaking motion is not an issue.
In general, drop in pore pressure and large spikes when soils accelerated downslope help soils in gaining shear strength and result in preventing soils from large displacements. Nevertheless, suddenly and temporary drop of downward soil accelerations, when base is accelerating upward, assists large soil displacements in liquefied slopes and also allows pore water to dissipate which lead to rapid drops of pore pressures.

3.5 Conclusion

Liquefaction was observed at around 0.9 s when excess pore pressure reached initial effective stress, the 25Hz was observed and shear wave velocity dropped to 20 m/s. Observed attenuation and amplification in acceleration has a substantial influence on slope performance when subjected to earthquake. Occurrence of soil column natural frequency results in sudden drops of acceleration and consequently caused relatively large displacement and drop of pore pressure. Decrease in pore pressure results in different effects when base of the model is accelerated upward and downward, in which displacement is aided and impeded, respectively.

The centrifuge modeling and the image processing technology is well used in observing liquefaction-induced lateral spreading during earthquake. The study allows us to pre-investigate the double cyclic behaviour which possibly does happen in real life, but are unable be seen because of the multi-frequency motion. If the double cycle nature does happen, it possibly lead to potentially damaging energy at natural frequencies of the system even if these natural frequencies are not present in the original earthquake motion.

Further study of the double-cyclic behaviour and its effect on foundation will benefit earthquake studies. In addition, full-scale study using a single frequency is required to develop an understanding of the behaviour.
Figure 3.1 Acrylic glass box
Figure 3.2 a) Plan and b) profile of lateral spreading model, (Model 1 in model scale)
Figure 3.3 Acceleration of a) the middle of loose sand layer (ACC3), b) the base of loose sand layer (ACC1) and c) the input motion (Model 1 in model scale)
Figure 3.4 Wavelet analysis of acceleration at a) the middle of loose sand layer (ACC3), b) the base of loose sand layer (ACC1) and c) the input motion (Model 1 in model scale)
Figure 3.5 Excess pore pressure  a) close to the surface of loose sand layer (PPT3) and  b) at the mid depth of loose sand layer (PPT1) (Model 1 in model scale)

Figure 3.6 Profile of total displacement due to lateral spreading a) Model 1 and b) Model 2, (in model scale)
Figure 3.7 Surface displacement (in model scale)
Figure 3.8 a) An acceleration time history of input motion, (b), (d), (f) and (h) horizontal displacement profile through slope and (c), (e), (g) and (i) normalized displacement profile through slope during time 1.04 sec to 1.09 sec (Model 1 in model scale)
Figure 3.8 a) An acceleration time history of input motion, (b), (d), (f) and (h) horizontal displacement profile through slope and (c), (e), (g) and (i) normalized displacement profile through slope during time 1.04 sec to 1.09 sec (Model 1 in model scale)
Figure 3.9 In-cycle soil responses including a) soil displacement, b) input motion, c) velocity and d) excess pore pressure (Model 1 in model scale)
Figure 3.10 a) A base acceleration and profile transient displacement due to lateral spreading in b) interval A, c) interval B, d) interval C and e) interval D (Model 1 in model scale)
Figure 3.10 a) A base acceleration and profile transient displacement due to lateral spreading in b) interval A, c) interval B, d) interval C and e) interval D (Model 1 in model scale)
Figure 3.11 Inferred shear wave velocity degradation with time (Model 1 in model scale)
Chapter 4
Dynamic Pile-Soil Interaction in Laterally Spreading Slopes

4.1 Introduction

Lateral spreading is the downslope inertial mass movement of gently-sloping deposits of liquefiable sands during earthquakes. Liquefaction-induced lateral spreading is known to be one cause of severe damage to deep foundation systems installed in sloping ground. One of the most widely referenced examples of lateral spreading is the earthquake triggered collapse of the Showa River Bridge during the 1964 Niigata earthquake in Japan. In this particular case study, the laterally spreading soil caused the piles supporting the bridge to deflect, increasing the distance between piers until such time as the decks were able to slip and fall from the piers. During the 1964 Alaskan earthquake, the 1989 Loma Prieta earthquake and the 1995 Kobe earthquake, lateral spreading also played a major role in the damage to civil engineering structures. Despite the potential for damage to be caused to piles by lateral spreading, much remains to be understood about the exact nature of this dynamic soil-pile interaction problem.

The majority of experimental data and literature on the topic of dynamic soil-pile interaction in liquefied ground comes from the centrifuge modeling community. Abdoun (1997) and Abdoun et al. (2003) conducted a series of dynamic centrifuge tests at Rensselaer Polytechnic Institute (RPI) to measure the bending moment in single piles and pile groups subjected to lateral spreading. Single piles and pile groups were tested in soil slopes consisting of two or three layers of differing soil properties. The results show that piles in ground containing a stiff layer at the surface show a larger bending moment compared to a ground with only soft layers. Goh and O’Rourke (1999) used this study as a basis to predict the shape of the relationship between lateral pressure applied to the soil (p) and soil-pile relative displacement (y); however, because the
transient profiles of soil deformation could not be measured in this study, the required displacement to produce residual state must be taken as approximate.

Wilson et al. (2002) also carried out dynamic centrifuge tests at the University of California (Davis) to investigate the behaviour of piles in liquefying ground with the aim of back-calculating the relationship between lateral pressure applied to the soil (p) and soil-pile relative displacement (y). The p-y relationships for the liquefied loose sand layers exhibited small lateral resistance despite being subjected to large relative displacements (on the order of 70 mm). In medium dense sand layers, soils provided more lateral resistance which the resistance-relative displacement ratio remains constant up to 15 sec and subsequently decreases. As well, hardening behaviour is observed when the relative displacement exceeds the maximum relative displacement in previous cycle. Based on this physical modeling data, Bradenberg et al. (2007) used a monotonic static “beam on nonlinear Winkler foundation” (BNWF) analysis in an attempt to predict the response of pile groups in the dynamic centrifuge tests. These analyses were performed by two separate methods: one in which limiting values of lateral pressure were applied to the pile (BNWF_LP), and one in which the free-field soil displacements were imposed on the free end of the soil springs in the models (BNWF_SD). When structural inertia forces were taken into account, BNWF_SD provided good prediction for stiff pile groups but slightly overpredicted the pile bending moment for more flexible pile groups. BNWF_LP provided good prediction of the pile bending moment for large deformation conditions but overpredicted the bending moment for small to medium earthquake motions in which the ground displacement is inadequate to fully mobilize the limiting lateral earth pressure.

Haigh (2002) conducted dynamic centrifuge tests at Cambridge University and observed suction spikes and large cycling of pore pressure near piles in liquefied sands. The amplitude of the near-pile pore pressure cycles were observed to be greater than that observed in the free-field, Haigh
(2002) hypothesized that this is likely due to the presence of the interface between the soil and pile that allows faster pore-pressure dissipation than in the free-field soil and causes high shear strains at this pile-soil boundary to initiate dilative soil behaviour local to the pile. In addition, Haigh (2002) inferred stress paths from PPTs and lateral total stress cells which measured the earth pressures at the front and back of a pile subjected to lateral spreading in the centrifuge. Stress paths were observed to follow the passive failure line on the upslope side of the pile, whereas the stress paths on the downslope side of the pile were observed to cycle between the active and passive failure lines. However, as the transient profiles of soil deformation could not be measured in this work, it was again not possible to infer p-y relationships from this experimental data.

To compliment these dynamic centrifuge modeling studies, Ashford et al (2006) and Juirnarongrit and Ashford (2006) carried out full scale experiments using blast induced laterally spreading ground to investigate the load transfer relationships between liquefied soil and pile foundations. The results of this study indicate that the bending moment and pile head displacement are independent of the free-field soil movement if the soil pressure is fully mobilized. In contrast, these quantities were found to be highly dependent on free-field soil movement if the soil pressure is not fully mobilized.

These past dynamic centrifuge modeling studies and the field work of Ashford et al (2006) indicate that knowledge of the full load-deformation relationship is required to fully capture the behaviour of piles in laterally spreading soil. In order to reduce the uncertainty surrounding this vital component of the design of pile foundations to withstand lateral spreading, this Chapter presents the results of a comprehensive study of three centrifuge tests performed to quantify the full p-y curve relationship of piles in liquefiable soils. This objective is made possible using recent advances in high-speed digital imaging and PIV image analysis which now enables
experiments to be performed in which the transient soil deformation profiles throughout the depth of the model can be quantified. Unlike previous studies, the soil-pile relative displacement (\( y \)) can be directly measured throughout the depth of the pile foundation. This chapter discusses the observed near-pile soil behaviour, including the role of the suction spikes seen in Chapter 3, and the full pile and soil response during shaking using data from particle image velocity (PIV), strain gauges, pore pressure transducers (PPTs), and accelerometers (ACCs).

4.2 Methodology

4.2.1 Centrifuge modeling
Dynamic centrifuge model tests were carried out using the C-CORE Geotechnical Beam Centrifuge in St. John’s Newfoundland. Three centrifuge models were tested, each of which had a submerged 6 degree infinite slope consisting of 200 mm of liquefiable loosely deposited sand overlaying a layer of dense sand. Four ICP miniature accelerometers (ACCs) and four GE DRUCK PDCR-81 pore pressure transducers (PPTs) were placed at selected depths on both the upstream and downstream sides of the model pile to observe the dynamic response of the soil at the front and back of the model pile. Additional details concerning the test configuration, soil materials, instrumentation, and model preparation techniques can be found in Section 3.2.

4.2.2 Instrumented pile
Each model had a pre-installed, highly-instrumented pile penetrating through the surficial loose liquefiable sand layer (Dr~40%) and was embedded 60 mm into the lower dense sand layer (Dr~80%). The experimental piles in the centrifuge experiments were model-sized reductions of a 0.5 m diameter concrete pile which is representative of the stiffness of bridge piles. In the first two models, the pile was installed at 200 mm from the left side and placed in the middle of the plain-strain width of the sample container. Hence, the pile was fully surrounded by soil and was minimally affected by the box boundaries. This configuration allowed a high speed camera to see
the free-field movement of the soil profile. The pile was made from an aluminum pipe which has a diameter of 9.625 mm and a thickness of 0.89 mm which then yield a bending stiffness of 101 MNm$^2$ in prototype scale. In the third model, a rectangular pile was chosen and placed against the front window in order to allow the camera to capture the movement of the pile and near-pile soil during the shaking. To obtain the bending stiffness near to the circular pile and an equal contact area against lateral spreading movement, the rectangular pile was made from an aluminum bar with a cross section area of 9.625 mm, x 6.35 mm (the 6.35 mm side faced the glass) as shown in Figure 4.1.

Ten sets of the full-bridge strain gages circuit were attached to measure the bending moment data along the pile length. The smallest size of strain gages using Vishay double grids model EA-06-060PB-350 which measured 3 mm in overall length and 1.651 mm in grid width was selected to ease gauge attachments. To waterproof and protect the strain gauges, the pile was then covered by heat shrink. Figure 4.2 illustrates the pile when fully instrumented with the strain gauges and when covered by heat shrink. The strain gages attached to the pile were calibrated using statically applied loads on cantilever beam system.

Despite the inevitable additional boundary effects from pile-window friction in the third model, this test enables a unique observation of the mechanism of soil-pile interaction to be captured by the high-speed camera. Thus, the results from the third test are used to illustrate the mechanism of soil-pile interaction while the results of the first and second tests are used to present the free-field soil behaviour and design-related values. As they give a unique understanding of the processes at work, the results from the mechanism test are discussed first.
4.3 Observed mechanism of soil-pile interaction

4.3.1 Soil accelerations

The input acceleration was monitored by ACCs attached to the base of the box while the soil accelerations were measured using ACCs placed at full and mid depth of the liquefiable layer during sample preparation. A servo-hydraulic earthquake actuator generated a simulated earthquake and transmitted the motion to the base of the box. Figure 4.3 (a) shows the input acceleration measured at the base of the box, in this case, 50 cycles of a 10 g amplitude, 50 Hz frequency sinusoidal wave. The motion is subsequently transmitted through the soil layer to the ground surface by the propagation of shear stresses. Because very dense sands are not susceptible to liquefaction and their shear stiffness remains largely unchanged when subjected to shear compare to liquefied sand, the acceleration was transmitted virtually unmodified in the dense-deposit layer. Thus, there is no significant differences between the input motion, Figure 4.3 (a), and the soil acceleration at the bottom of liquefiable layer, Figure 4.3 (d), (e). During the second half of the shaking, the input acceleration was attenuated and amplified at the mid-depth level of the liquefied layer on the upslope of the pile as in Figure 4.3 (b). In contrast to the upslope sensor, the acceleration data of the ground surface at the downslope of the pile shows the attenuation and amplification of the input acceleration occurring at a few cycles earlier, Figure 4.3 (c).

As discussed in Chapter 3, the decrease in stiffness and strength of the liquefied sands reduced the sand’s ability to transmit the shear stress from the dense lower layer to the loose liquefied upper layer. Hence, the input acceleration was attenuated through the liquefied sands. An appearance of the high frequency acceleration in liquefied sands results in the amplified, sharp spikes. The observation that the liquefied sand attempts to tune itself to the soil column’s natural frequency causes this co-seismic behaviour.
4.3.2 Excess pore water pressures

Four pore pressure transducers (PPTs) were placed at shallow and mid depth of the liquefiable sand layer during sample preparation to monitor pore pressure generation and dissipation in the soil layers. PPT data is used to measure the degree of liquefaction and indicate the loss of strength during earthquake shaking. Furthermore, the forces imparted on the pile can be approximated from the pore pressure profile together with the total stress profile at the back and front of the pile.

Excess pore pressures generated at the soil at depth 40 mm from the surface during the tests at the upstream and downstream sides are plotted in Figure 4.4 (a) and (b), respectively. The excess pore pressures increased slightly in every cycle until the end of the motion and negative spikes was observed in both upslope and downslope area. The negative excess pressure measured at downslope side is however larger than observed in upslope side.

4.3.3 Slope deformation

The transparent acrylic-sided box used in the experiment facilitates a visual study of laterally spreading ground. Placing the model pile against the transparent wall allowed for an observation of how sand at the front and back of the pile responded to an earthquake and pile movement. However, having one side of the pile against the wall causes additional boundary effects: the appearance of pile-glass friction, the plane of symmetry and a potentially higher pore pressure dissipation rate at the sand-glass interface. In laterally spreading ground, soil movement is greater than pile movement and its difference generates forces which are imparted onto the pile. Beside inertia forces from earthquake-induced pile movement and soil movement, pile deflections highly depend on the pile’s end conditions. To avoid the complexity of pile end conditions, a single pile with free end conditions was selected.
Figure 4.5 shows the results of the GeoPIV image analysis of the total post-earthquake deformation observed in both Model 1 and Model 3. The zone of laterally spreading soil observed in the mechanism test is similar in terms of depth to that observed in the free field, but with a noticeable reduction in horizontal movement due to the presence of the pile. This reduction in horizontal displacement is also clearly visible in deformation field of the mechanism test (Figure 4.5a) by comparing the total displacements immediately upstream and downstream of the pile near the surface of the liquefied soil deposit.

In order to more fully describe the evolution of soil displacements in the near-pile region, soil deformation relationships for three different depths have been selected from the front and back side of the pile as shown in the inset of Figure 4.5 for further investigation. The first depth of 7 mm from the ground surface (PIV patches UP1 and DOWN1) is the closest area to the surface that the GeoPIV software could feasibly track because the soils are extremely altered in visual appearance during the earthquake shaking. The two other depths chosen were 37 mm (patches UP2 and DOWN2) and 67 mm (patches UP3 and DOWN3) from the ground surface, in which the significant displacement were observed. Figure 4.6 (a) shows a comparison between the cyclic displacement paths of patches UP1 and DOWN1. These results clearly show the effect of the presence of the pile on the deformation of soil upslope of the pile. Patch DOWN1 started with zero displacement at the beginning and flowed cyclically downstream until it reached 7 mm of horizontal displacement and 17 mm of vertical displacement. The similar PIV patch at the upstream, UP1, initially showed the same cyclical response during early cycles, but soon deformations were limited by the pile to 2 mm horizontal displacement. This patch then continued to experience displacement cycles, but this time with a slow accumulation of vertical displacement and no additional horizontal movement. The results are shown in Figure 4.6 (b) and (c), for the other two depths. These plots confirm that the magnitude of both horizontal and
vertical deformations decrease with depth into the liquefied soil. As a result, the relative soil pile
deformation will similarly be a function of depth.

4.3.4 Pile - free field soil movement
The difference in total post-earthquake displacement observed in both the mechanism centrifuge
test (RK5) and the free field response of centrifuge test RK3 is presented in Figure 4.7. The free-
field displacement splits from the pile displacement at the elevation of 240 mm and then the
difference continues to increase. The pile head’s displacement is around 4 mm, which is
equivalent to 20 cm in a prototype scale. Also, free-field displacement measured at the surface is
around 30 mm, which corresponds to 1.5 m at full scale.

4.3.5 Pile response
The response of the pile to this free field lateral spreading displacement is captured at the end of
the earthquake in Figure 4.8 in terms of pile displacement, rotation, and distributions of bending
moment, shear force and distributed load on the pile with depth. The strain gages on the highly
instrumented pile yield in measurements of bending moment. The displacement and rotation are
calculated from integrating this bending moment data whereas shear and net lateral pressures are
calculated from differentiating the bending moment data. The sign convention for displacements
is positive downslope. Positive bending moments create a concave down shape in the pile and
positive net lateral pressures imply that soil pressure acting downslope on the pile is more than
upslope.

Due to the flow of lateral spreading soil during the earthquake the model pile experienced a total
displacement of approximately 4 mm downslope. A calculation of the rotation of the pile
indicates that the pile rotated downslope about a point slightly higher than the base of the pile, but
still within the dense sand layer. The maximum bending moment at the end of earthquake shaking
was approximately 2 N-m (corresponding to 250 kNm at prototype scale) and was located at an
elevation of 200 mm. This elevation corresponds closely to the interface between the loose and dense soil layer.

Since the model pile was located at the window, the deflected shape could also be observed using geoPIV image analysis to investigate and confirm the internal consistency of the data set. Continuous lines in Figure 4.8 represent the results obtained by calculation from the strain gauge bending moment data while the dashed lines present the observed and derived profiles from the geoPIV image analysis. These two different measurement techniques yield nominally identical displacement and rotation profiles. However, as the image processing data must be differentiated one and two additional times for the derived shear force and distributed load distributions, respectively, it is not unexpected that these profiles contain some oscillation about the trends calculated from the strain gauge data. Rather than attempt to further process the imaging data to isolate and minimize this oscillation, net lateral pressures will be derived from the strain gauge data. The quantitative confirmation of the pile behaviour from these two completely independent measurement techniques confirms the internal consistency of the data stream from the high speed camera and the data acquisition system.

Figure 4.9 demonstrates the typical transient pile response throughout one full cycle of shaking. This cycle was taken from approximately 1.45 seconds to 1.48 seconds of the earthquake record (i.e. near the end of the shaking) and is subdivided into 6 time intervals for the ease of discussion: A, B, C, D, E and F. At the A and F intervals, the base of the slope model is accelerating downslope. As a result, the liquefied soil attempted to flow backwards towards the upslope direction. Because of this reversal of acceleration, the downslope lateral pressures acting on the pile and the resulting bending moments in the pile are reduced. One potentially useful way of visualising this phenomenon is to consider the direction of the acceleration vector acting on the box relative to the slope angle of the soil. At the peak horizontal acceleration of 10g, with a static
centrifugal acceleration of 50g, the resultant acceleration vector is angled at 11 degrees upslope of vertical. The 6 degree slope thus has an equivalent slope of 5 degrees in the upslope direction (i.e. 11 degrees minus the six degree angle of the slope). Conversely at the opposite phase in the cycle the equivalent slope angle is 17 degrees downslope (11 degrees plus the six degree angle of the slope). During the B interval, the base acceleration changed its direction from downslope to upslope. Intervals B and E exhibited similar maximum bending moments of 1.1 N-m at 190 mm while E showed larger bending moments in the top half of the loose soil layer. Line C shows the bending moment just before soil acceleration reached its maximum value whereas D shows the bending moment that just after a peak of acceleration. Similarly to B and E, backward soil flow in D caused a higher bending moment nearer to the surface than soil flowing forward in C. Throughout the entire cycle, therefore, the maximum transient bending moment is near the interface between liquefied and non-liquefied soil and this region always experienced positive bending moments. Near the surface, the bending moments were observed to be both positive and negative depending on the direction of basal acceleration.

The bending moments measured by the strain gauges on the pile are plotted versus time in Figure 4.10 for a few selected depths. The magnitude of pile bending moments was observed to increase during the early cycles of the earthquake at all depths. The moments at depths of 80 mm and 140 mm were then observed to reach a peak value before reducing in magnitude with further cycles. It is interesting to note that this occurred earlier at the shallower location on the pile and later in the earthquake record for the deeper location. The presence of these peak values and the relative timing of their reduction in magnitude can be explained by the observation that the maximum bending moments in piles tend to occur at the boundary between soft liquefied soil and stiff non-liquefied soil (as seen in this test and in earlier studies such as Abdoun, 1997). It is therefore logical that the bending moment at each elevation was maximized at the time this elevation of the pile was coincident with the transient boundary between liquefied and non-liquefied sand. As the
earthquake progressed, the liquefaction front propagated deeper from the surface, and so to the location of higher bending moments.

4.4 Load-displacement relationship

Load-displacement relationships (i.e. p-y curves) are generally the method of choice to design piles against lateral loading. Although recommendations for p-y curves for laterally loaded piles are provided by design manuals and specifications, the complexity of load and deflection cycles makes the generation of a dynamic p-y curve for piles in laterally spreading ground impossible. Instead, a pseudo-static backbone p-y curve is generated from the peaks of lateral load to err on the side of over estimating net lateral earth pressure so that the design method will yield conservative estimates of pile bending moment requirements.

Unlike the other figures shown in this chapter in which the primary source of data was obtained from the “pile at the window” mechanism test (Model 3, test RK5), the data from Model 2 will be used to develop p-y curves in order to minimize the influence of wall friction on the pile response. Figure 4.11 present the relationship of net lateral pressures and dimensionless displacements at the nine different gauge locations. As shown in this figure, gage one is located near the base of the pile and gauge nine is located near the soil surface. The net lateral pressures (p) and pile displacements (y_p) are calculated from the bending moment data whereas the displacements of soil at free-field (y_s) were obtained from the geoPIV image analysis. The free-field displacements were subtracted from the pile displacement and subsequently normalized by the pile diameter to produce a dimensionless relative displacement. The peak loads and minimum loads in the plots were selected from the peak and minimum loads observed within 1 cycle and are represented by circles and diamonds in Figure 4.11, respectively. Since the objective is to create a pseudo-static backbone curve of peak net lateral pressures, the peak loads during the cycle (i.e. the circles) are of primary importance.
It has to be noted that at this section the dimension and data are described in prototype scale to make easier for design purposes.

The experimental p-y curve observed at 1 m depth shows that peak lateral stresses increase to 50 kPa when the dimensionless displacement are within 0.5. The lateral stresses then dropped when the dimensionless displacement increased and, after the displacement passed 2, the net lateral pressures decreased from 25 kPa at a dimensionless displacement 2 to a value of 10 kPa by the time the lateral spreading event ceased. At depth 2.5 m from the ground surface, a similar experimental p-y curve was observed except that there were two distinct levels of peak net lateral pressures observed: one in each alternate cycle. The observed p-y curves at depths of 4 m and 5.5 m yielded maximum net lateral pressures of 80 kPa and 130 kPa. Again, two separate sets of peak lateral stresses were also found in alternate cycles. The difference between these peaks was particularly evident at the depth of 5.5m in which the difference between these two sets of peak lateral stresses was 40 kPa. It should be noted that 5.5 m depth is immediately above the shear zone observed in the free-field displacement of the liquefied slopes. At soil layers deeper than 5.5 m, negative net lateral pressures were observed with little deformation recorded which is consistent with this transition of behaviour from above to below the shear zone.

In order to examine the effect of increasing of pore water pressures on the lateral resistance, excess pore pressure ratios, \( r_u \), were also plotted against dimensionless displacement in Figure 4.12. The p-y curve for a depth of 1.25m is presented alongside the excess pore water pressure ratio in Figure 4.12 (a) – (b), respectively. As the soil progressively became more liquefied, the net lateral pressure acting on the pile was observed to decrease. The same comparison is also made in Figure 4.12 (c) – (d) for a depth of 3.75 m. At this depth, the shaking motion was not strong enough to cause excess pore pressure in soils at 3.75 m depth to equate the effective overburden stress and the soil was therefore not expected to be fully liquefied. Instead, the soil
quickly attained an $r_n$ value of approximately 0.8 which it maintained for the duration of shaking. As a result, the peak lateral earth pressure acting on the pile remained relatively constant until the motion stopped.

4.5 Discussion

The soil-structure interaction behaviour observed in this experimental study indicates that the presence of piles clearly obstruct the soil deformation profile from the free field. This relative soil-pile deformation causes shear stresses to be induced in soil in the region surrounding the pile which can cause dilative behaviour of the liquefied soil. This leads to the negative pore-pressure spikes seen in the data, which in turn lead to an increase in soil strength and hence to both a decrease in soil displacement and an increase in the net lateral pressures exerted on the pile. From the cyclic displacement path and profile of total slope displacement, it can be seen that the flowing liquefied soils upslope of the pile were restrained by the pile while the down slope was largely free from pile restraint. It is this restraint of the liquefied soil downslope displacement which is the major cause of the lateral stresses exerted on the pile during lateral spreading as the pile must provide sufficient restraining force to the upslope soil to replace the upslope shear stresses which were lost during liquefaction. The effect of the restraint can also be seen in the difference of acceleration and pore pressure responses in upslope and downslope sides of the pile.

The double cycle behaviour observed in Chapter 3 was seen to cause two different levels of peak lateral loading. To describe the effect of this double cycle behaviour on a pile, the peak net lateral pressures are plotted in Figure 4.13 (a) while the peak loads on the alternate cycles are plotted in Figure 4.13 (b). These figures clearly indicate the much higher lateral stresses which can be expected if the soil column experiences this phenomenon. For example, the maximum net lateral pressure at the depth of 5.5 m depth is reduced from 130 kPa to 90 kPa when the double cycle behaviour is neglected. Data without the effect of the double cycles (i.e. Figure 4.13, b) shows
remarkably consistent behaviour in which the peak net lateral pressure is not a strong function of depth within the liquefied soil with peak values ranging between 40 kPa and 90 kPa.

The pseudo-static p-y curve relationships observed in the liquefied layers sand layers exhibit a reasonable degree of fit with a 4th degree polynomial through the p-y curves data observed in liquefied sand (Figure 4.14). These p-y curve relationships agree well with the previous data developed by Abdoun (1997) and Haigh (2002) for both the peak and residual lateral stresses which can be expected in laterally spreading soil. Because these previous studies have been unable to measure the local dimensionless relative displacement at the peak load, the displacement values at the peak lateral stress results of Haigh (2002) are denoted as an unknown in the figure.

The experimental p-y curve relationships are also compared to the assumed p-y curve relationship for liquefield sand of Goh and O’Rourke (1999) in Figure 4.14. As the relationship of Goh and O’Rourke (1999) is in terms of undrained shear strength, the peak values of force have been matched to enable a comparison of the mobilisation displacement of the p-y curve. The assumed p-y curve from Goh and O’Rourke (1999) shows much stiffer p-y response in which the peak and residual net lateral pressure are expected at 0.0125 and 0.15 y/D whereas our tests results show that 0.5 and 2.0 y/D is required to generate the peak and residual net lateral pressure, respectively. In the light of this finding, a prudent design decision would be to extend the peak value of net lateral pressure to a displacement of 1 y/D or greater before allowing a drop to residual.

It has to be noted that the proposed shape of p-y curve is calculated from testing the single pile with no constraint and weight applied at the top of the pile in centrifuge. The additional inertia effect from superstructure mass, boundary effect and end conditions must be concerned to obtain accuracy.
4.6 Conclusion

Dynamic centrifuge experiments were carried out to investigate the behaviour of soil surrounding piles and the pile performance in liquefaction-induced laterally spreading ground. Various instruments were used to record and measured the responses of the pile and slopes; in particular, the use of a high-speed camera and image processing technology has enabled the measurement of relative soil-pile displacements with depth as the earthquake progresses for the first time.

The results of the analyses of these images were shown to agree well with the stain gauge data, whilst it was observed that obtaining smooth data is difficult when using high order differentiation, as high frequency components in the initial data tend to dominate the results.

Due to the difference in soil and pile response to earthquake shaking, piles embedded in slopes susceptible to lateral spreading will resist the downslope cyclical flow of the liquefied soil. This restraint will lead to a transient net lateral pressure acting on the pile.

The observed soil-structure interaction relationships, as quantified by pseudo-static p-y curves, agree well with the previous data developed by Abdoun (1997) and Haigh (2002) for both the peak and residual lateral stresses which can be expected in laterally spreading soil. The peak net lateral pressure was observed to not be a strong function of depth within the liquefied soil and peak values ranging between 40 kPa and 90 kPa. These peak values were observed to be mobilized at significantly larger displacements than predicted by Goh and O’Rourke (1999). Finally, the double cycle behaviour discussed in detail in Chapter 3 was observed to greatly increase the transient net lateral pressures imparted to the soil structure.
Figure 4.1 Cross section area of a) circular and b) rectangular model pile

Figure 4.2 Model instrumented pile without heat shrink and with heat shrink
Figure 4.3 Acceleration of a) the input motion, the middle of loose sand layer at b) upstream side and c) downstream side, and the base of loose sand layer at d) upstream side and e) downstream side (Model 3 in model scale)

Figure 4.4 Excess pore pressure at the 40 mm depth soil at a) upstream and b) downstream (Model 3 in model scale)
Figure 4.5 Profile of total displacement due to lateral spreading of a) soils around the pile and b) free-field soils (Model 3 and Model 1 in model scale)
Figure 4.6 Cyclic displacement of soils around the pile at the depth of a) 7 mm, b) 37 mm and c) 67 mm from the ground surface (Model 3 in model scale)
Figure 4.7 Pile-free field soil displacement (Model 3 and model 1 in model scale)
Figure 4.8 Pile responses of a) displacement, b) rotation, c) bending moment, d) shear, and e) lateral pressure when at the time of 1.47 seconds (Model 3 in model scale)
Figure 4.9 Profiles of in-cycle pile displacement and bending moment (Model 3 in model scale)
Figure 4.10 Bending moment time-history at different depth (Model 3 in model scale)
Figure 4.11 Load-displacement relationships at selected depth (Model 2 in prototype scale)
Figure 4.11 Load-displacement relationships at selected depth (Model 2 in prototype scale)
Figure 4.11 Load-displacement relationships at selected depth (Model 2 in prototype scale)
Figure 4.12 Comparing a) lateral pressure and b) excess pore pressure ratio at 1.25 m depth and c) lateral pressure and d) excess pore pressure ratio at 3.75 m depth (Model 2 in prototype scale)
Figure 4.13 Backbone curve of P-Y plot (Model 2 in prototype scale)
Figure 4.14 Comparing results of backbone plot with literature (Model 2 in prototype scale)
Chapter 5
Conclusions

5.1 Evolution of deformation in laterally spreading slopes
Despite recent tangible progress towards understanding seismic soil-pile interaction, significant uncertainties regarding the design of piles subjected to lateral spreading remain. The work described in this thesis was divided into two objectives to address these uncertainties. The first objective was to study the free-field mechanism of lateral spreading through the use of novel experimental technology consisting of high speed imaging and geoPIV image analysis of centrifuge tests. For the first time, the evolution of soil displacement during lateral spreading was observed through the profile of a dynamic centrifuge model. This data enabled a detailed description of the soil displacements including the in-cycle, surface, soil profile and slope profile displacement.

In these model tests, liquefaction of the soil occurred when excess pore pressure approached the initial effective stress, and as a consequence, the shear wave velocity of the soil column dropped. This reduction in the shear wave velocity then was observed to excite the natural frequency of the soil column which resulted in the appearance of low frequency acceleration components and consequently caused large suction spikes of negative excess pore water pressure. These analyses indicate that the propensity for the soil to “auto tune” to the natural frequency of the soil column may transform the cyclic deformation of the slope. In this manner, this study confirms the presence of these “double cycles” observed by Haigh (2002) and explains their significant role in the development of lateral spreading displacements.

5.2 Pile-soil interaction in laterally spreading slopes
The second objective of the study was to investigate the interaction of laterally spread soils and pile foundations to produce design recommendations regarding the appropriate p-y curve
applicable to piles in liquefiable soils. This objective was met by performing a series of dynamic centrifuge model tests taking advantage of recent advances in digital image analysis and high speed photography to measure the transient deformation response of soil in a laterally spreading slope. Due to the difference in soil and pile response to earthquake shaking, piles embedded in slopes will restrain the flow of liquefied soil. This restraint leads to the development of transient lateral stresses on the pile.

The observed soil-structure interaction relationships, as quantified by pseudo-static p-y curves, agree well with the previous data developed by Abdoun (1997) and Haigh (2002) for both the peak and residual lateral stresses which can be expected in laterally spreading soil. The peak lateral pressure was observed to not be a strong function of depth within the liquefied soil and peak values ranging between 40 kPa and 90 kPa. These peak values were observed to be mobilized at significantly larger displacements than predicted by Goh and O’Rourke (1999). Finally, the double cycle behaviour discussed in detail in Chapter 3 was observed to greatly increase the transient lateral pressures imparted to the soil structure. Ignoring these effects is therefore unconservative.

5.3 Implications for engineering practice

The model test data indicates the potential for the natural frequency of the soil column to introduce potentially damaging energy at natural frequencies of the soil column even if these frequencies are not present in the original earthquake motion. Ignoring the effect of these frequencies may lead to the underprediction of lateral pressures on pile foundations and could potentially cause foundation failure. It is therefore recommended that the potential for “auto tuning” be anticipated in design, and that the proposed limiting p-y curve of Figure 4.13 be used as a design recommendation to predict the bending moment in piles subjected to lateral spreading ground deformation.
References


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Appendix A

Comparing Data of 3 Models

This study contains three centrifuge models in which the first two models were used in free-field investigation whereas the third model was used in soil-pile interaction observation. The models are expected to reproduce similar earthquake events except at the location of the piles. The repeatability of the centrifuge models are determined by three components including (1) CPT examining the soil layer profile; (2) Earthquake shaking as an input motion; (3) Responses of the model to the applied based motion.

CPT data measured before and after the models subjected to sinusoidal input motion are considerably consistent regarding to tip resistant and relative density calculated from equation proposed by Bolton (1993:1999) as shown in Figure A. 1 (a) and (c) respectively. However, the differences exhibited at the shallow soils when tip resistances are normalized and relative densities are calculated from equation recommended in Juang and Huang (1993) and Schmertmann (1978) with data provided by Olsen and Stark (2003) as shown in Figure A. 1 (b) and (d) respectively. The CPT data measured after the first earthquake is also shown in Figure A. 2.

Although, the slope characteristics are considerably similar, the gap between pile and container are likely to limit the liquefaction around pile foundations adjacent to the container boundary in model 3. Therefore, soil responses in model 3 are different from responses monitored in Model 1 and Model 2 which can be seen in acceleration time history from Figure A. 3, Figure A. 4 and Figure A. 5. Similarly, observed pile responses are less in Model 3 compared to Model 2 as shown in Figure A. 6. It has to be noted that the pile responses in the first model are incompetent due to occurring of the short-circuited in the strain gage systems during the tests.
Although dissimilar responses in Model 3 are observed due to boundary condition of the pile and container, it is expected. It is, therefore, reasonable to use free-field data in the first or second test to represent the free-field behaviour in the third model in qualitative and quantitative perspective. Nevertheless, the pile bending moment data observed in the third test is appropriately used only when involving the behaviour and mechanism aspects excluding results involving numerical data.
Figure A. 1CPT Data including a) cone Tip resistance, b) normalized tip resistance, c) relative density, (Bolton, 1999), and d) relative density, (Juang and Huang, 1993), measured prior to the dynamic centrifuge tests - sinusoidal motion.
Figure A. 2 CPT Data including a) cone Tip resistance, b) normalized tip resistance, c) relative density, (Bolton, 1999), and d) relative density, (Juang and Huang, 1993), measured after to the dynamic centrifuge tests - sinusoidal motions.
Figure A. 3 Acceleration of input motion
Figure A. 4 Acceleration of upslope soils during lateral spreading
Figure A. 5 Acceleration of downslope soils during lateral spreading
Figure A. 6 Pile responses during lateral spreading
Appendix B

Centrifuge Test Results of 2\textsuperscript{nd} Earthquake Shaking

As the model has already been exposed to the sinusoidal ground motion, the initial states of RK1 and RK2 are different. Firstly, compaction of the loose layer during shaking caused the sand to be denser at the beginning of RK2. Secondly, lateral spreading in the sample box resulted in the lower angle of slope inclination. Finally, the COSTA-A motion has a relatively low energy content being the earthquake event with probability 2\% within 50 year period in BC, Canada, compared to the sinusoidal event.

The COSTA-A motion can be seen from Figure B. 1 to have a peak acceleration of 16.1 g and a dominant frequency of 45.5 Hz. The attenuation of acceleration can be seen from the FFTs in which the Fourier amplitude at the dominant frequency diminished from 3600 g/Hz at the base to 3400 g/Hz at the base of the liquefiable layer and 3300 g/Hz at mid-depth in the liquefiable layer. By comparing wavelet plot of soil acceleration in Figure B. 2 (a) and (b) with the wavelet plot of the input motion input in Figure B. 2 (c), amplification of lower frequencies can be seen clearly in the acceleration at soil surface as well as the addition of higher frequencies. Figure B. 3, shear wave velocity can be seen to have decreased to 10 to 20 m/s, but this only happens for a short period when peak acceleration has occurred.

As a result of base shaking, the soil liquefied at the surface yet the generated motion was not strong enough to make the liquefaction propagate to mid-depth in the liquefiable layer, as shown as excess pore pressure readings in Figure B. 4. Excess pore pressure at the surface area increased rapidly from zero to 10 kPa within less than 0.1 s and oscillated slightly around 10 kPa. In the mid-depth area, the increasing trend continued reaching over 40 kPa at 0.75 s and then dropped. Moreover, the negative excess pore pressure spikes monitored near the surface during sinusoidal
motion are hardly seen in COSTA-A motion compared to what was observed with the sinusoidal motion.

Slope displacement and surface displacement are presented in Figure B. 5 and Figure B. 6. As was seen with the excess pore pressure data, area of large deformation are limited to the surface where less than 1 mm deformation was seen. Although, strong motion midway through the earthquake caused the large deformation, after peak various cycle with lower energy produce gradual displacement but for long period which end up with substantial displacement.

Because of the multi-frequency motion of COSTA-A motion, co-seismic behaviour is not as easy to analyze as with sinusoidal motion and hence this data is not plotted.
Figure B. 1 Acceleration of a) the middle of the loose sand layer (ACC3), b) the base of loose sand layer (ACC1) and c) the input motion (Model 1 in model scale)
Figure B.2 Wavelet analysis of acceleration at a) the middle of loose sand layer divided by the input motion (ACC3/input motion), b) the base of loose sand layer divided by the input motion (ACC1/input motion) and c) input motion (Model 1 in model scale)
Figure B. 3 Inferred shear wave velocity degradation with time (Model 1 in model scale)

Figure B. 4 Excess pore pressure a) close to the surface of loose sand layer and b) at the mid depth of loose sand layer (Model 1 in model scale)
Figure B. 5 Profile of total displacement due to lateral spreading (Model 1 in model scale)

Figure B. 6 Surface displacement (Model 1 in model scale)