GROUND MOVEMENTS DURING TUNNELLING IN SAND

by

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Abstract

During soft ground tunnel construction, if the face pressure of a tunnel boring machine is not strictly controlled, excessive ground movements will propagate vertically upwards causing significant damage to adjacent buried infrastructure and surface structures. In order to investigate the face pressure - ground deformation relationship for tunnels in sands, the construction process was modelled using the technique of geotechnical centrifuge modelling and the resulting ground deformations were recorded using digital image correlation. In these tests a unique tunnel face boundary condition was developed which allowed the boundary condition to be initially set as a zero strain condition before it was transformed into a load-controlled boundary to investigate the instability of the face. Tests were performed at four different burial depths in dry sand, corresponding to cover depths of 0.5, 1, 1.5, and 2 times the tunnel diameter. These results indicate that the face pressure at failure is largely independent of burial depth over the values tested. The ground deformation at the onset of tunnel face instability was found to be very small, and once initiated, the zone of ground deformations was observed to propagate upwards in a narrow chimney in front of the tunnel until it reached the ground surface causing subsidence. Further tests investigated the variation in ground deformations to be expected if a tunnel were to be passing through more complex ground conditions, including unsaturated sand, saturated sand, and the unique case of sand / clay mixed ground conditions. Ground deformations at tunnel face instability were much lower for the case of unsaturated sand, than for either the saturated or dry cases which showed broadly similar responses. In the mixed ground condition of a clay layer over topping a sand layer, the clay layer was found to only influence the tunnel face pressure – deformation response if the bottom of the clay layer was closer than 0.5 diameters above the tunnel crown.
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Chapter 1

Introduction

1.1 Tunnelling in urban areas

As urban expansion and renewal increases, the decrease in available land has increased the need for new transportation and service networks (e.g., water mains, gas pipelines and telecommunication and electric power networks) to be placed underground and in close proximity to existing infrastructure. In many cases, the tunnels for these networks must be constructed in soft ground conditions such as sands (both above and below the groundwater table), in clays, and in mixed ground conditions. The inherent problem associated with underground construction in soft ground conditions is that the underground excavations will alter the stress fields in the ground around the tunnels and deformations will occur. If these deformations are not strictly controlled during the construction process, excessive ground movements will propagate upwards potentially causing significant damage to adjacent buried infrastructure and surface structures.

Today, tunnelling in soft ground is commonly carried out by mechanized tunnel boring machines (TBM). There are two types of TBMs: slurry shields and earth pressure balance shields (EPB). Slurry shields work on the principle where the tunnel heading is excavated by cutting teeth and is supported by pressurised slurry, usually consisting of bentonite, which is circulated so that it and the excavated soil (called spoils) are removed to a separation plant. Likewise, with EPB shields, face support is obtained by retaining spoils in a working chamber so that sufficient confining pressure is reached. These machines work on the principle that ground deformations can be greatly reduced if the tunnel face is excavated while being supported by a positive face pressure. Thus, if the TBM can maintain support to the tunnel face, ground displacements can be
minimised until the soil stresses can be taken by the concrete tunnel lining segments constructed behind the TBM.

In theory, if the TBM could perfectly balance the earth pressure acting on the face of the excavation, no deformation should occur at the face. However, the higher the face pressure, the slower and more expensive the construction becomes (Mair and Taylor, 1997). Experience has shown that reduced face pressures can be successfully used to construct tunnels in soft ground without excessive ground deformations (e.g. Peck, 1969, Mair and Taylor, 1997). However, the extent of the ground movements associated with these lower pressures is unknown; it is necessary to know the face pressures at which ground movements are at a minimum to ensure safe and efficient tunnel excavation and construction using TBMs. Thus, accurate predictions of tunnelling-induced ground movements associated with differing face pressures in soft ground are essential for an efficient construction process which safeguards adjacent structures and utilities from damage.

1.2 Objectives

The objective of this thesis is to better understand the ground movements associated with tunnel construction; in particular:

- To develop a reliable, well-controlled centrifuge model in which the digital image correlation technique of GeoPIV (White et. al., 2003) can be used to track the face pressure – ground deformation relationship throughout the entire longitudinal cross-section of the tunnel model;

- To determine the influence of tunnel depth on the progression of ground movements with loss of tunnel face pressure for tunnels in dry sand. The results of the dry sand tests will be compared to previous research performed by Chambon and Corté (1989, 1991, 1994);
• To provide new experimental data describing tunnelling induced deformation in unsaturated sands;

• To establish the influence of saturated conditions on ground movements caused by tunnelling in sands below the water table; and

• To present the first ever experimental data describing the tunnelling-induced ground deformation for the specific mixed ground condition of tunnelling through saturated sand beneath a layer of clay.

1.3 Dissertation Structure

The dissertation is presented in manuscript format in four chapters. Chapter 2, the first manuscript, reviews the current state of knowledge regarding the ground deformations to be expected when tunnelling in sand, and describes the development of an experimental program to quantify the progression of ground deformations with loss of tunnel face pressure. Results are presented for the cases of both dry and unsaturated sand.

Chapter 3, the second manuscript, extends this work to investigate the ground deformation response in saturated ground conditions as well as the mixed ground case of tunnelling in a saturated sand beneath a clay layer.

Chapter 4 summarises the conclusions of the experimental study and provides suggestions for future research.
Chapter 2
Progression of Ground Deformations with Loss of Tunnel Face Pressure in Sand

2.1 Introduction
In densely populated urban areas, new transportation and service networks (e.g., water mains, gas pipelines and telecommunication and electric power networks) must typically be placed underground and in close proximity to existing infrastructure. As the tunnels for these networks are constructed, these underground excavations will alter the stress fields in the ground around the tunnels and deformations will occur. If these deformations are not strictly controlled in the construction process, excessive ground movements will propagate upwards causing significant damage to adjacent buried infrastructure and surface structures.

Today, tunnelling in soft ground is commonly carried out by mechanized tunnel boring machines (TBM). These machines work on the principle that ground deformations can be greatly reduced if the tunnel face was excavated while being supported by a positive pressure. Using a screw auger, the soil which is excavated by the cutting teeth of the TBM, called spoils, can be transported to the back of the TBM and disposed of without losing face pressure. Thus, if the TBM can maintain support to the tunnel face, ground displacements can be minimised until the soil stresses can be taken by the concrete tunnel lining segments constructed behind the TBM.

In theory, if the TBM could perfectly balance the earth pressure acting on the face of the excavation, no deformation should occur at the face. However, the higher the face pressure, the
slower and more expensive the construction becomes (Mair and Taylor, 1997). Experience has shown that reduced face pressures can be successfully used to construct tunnels in sands without excessive ground deformations (e.g. Peck, 1969, Mair and Taylor, 1997). Thus, accurate predictions of tunnelling-induced ground movements associated with differing face pressures are essential for an efficient construction process which safeguards adjacent structures and utilities from damage.

### 2.2 Review of previous work

Since field deformation monitoring is an essential component of the tunnel construction process in the urban environment, considerable data has been reported in the literature on observed ground deformations during these projects. From this data, Mair and Taylor (1997) have produced a state-of-the-art report that summarises the literature to date regarding tunnelling in the urban environment, where particular emphasis was given on ground movements associated with tunnel construction and the modelling and prediction of ground movements. Notable publications reviewed by Mair and Taylor (1997) are Peck (1969), Mair (1979), Chambon and Corté (1989, 1991, 1994), and Leca and Dormieux (1990). The findings of these researchers are discussed briefly in this section.

Based on numerous case studies, Peck (1969) presented at that time, a state-of-the-art report that suggested three important requirements for the satisfactory construction of a tunnel. The first is related to stability, as the construction method must be chosen to build the tunnel safely, with particular attention paid to stability of the tunnel heading, or face, prior to installation of the tunnel lining. Secondly, the tunnel excavation and construction should not cause any ground movements where any unwanted damage to adjacent structures, utilities, and roadways occurs.
Thirdly, during the design lifetime of the tunnel the lining (whether temporary or permanent) should be able to withstand any influence to which it will be subjected to.

Of particular importance to the tests reported in this chapter, a great deal of physical research has been performed regarding the modelling of stability and prediction of ground movements due to tunnelling. Full-scale physical tests with varying parameters would be too difficult and expensive to undertake, and small scale tests at 1 g (though more cost effective and with the ability to reproduce with ease) do not realistically model real world stresses in the ground. This is especially important since self weight of the ground that is being tunnelled through is a major influencing factor of tunnel stability and subsequent ground movements; small scale geotechnical centrifuge model tests are appropriate since they are able to model real world stresses in the ground.

Much physical research has been performed regarding the ground movements associated with tunnelling in clays. Some of the earliest centrifuge tests were performed by Mair (1979), who performed centrifuge modelling research to examine tunnel collapse in soft clay. He found that the failure mechanism propagation in soft clay is observed to propagate upwards and outwards from the tunnel invert by several times the tunnel diameter. This creates a very broad settlement trough on the ground surface, and has been further confirmed by physical research by Grant and Taylor (1996, 2000) for tunnels in moderately stiffer clays, where the use of digital image analysis was used.

However, to date, there has been limited work done on the centrifuge modelling of tunnels in sandy soils. The earliest centrifuge work examining the relationship between face pressure and
face stability was investigated by Chambon and Corté (1989, 1991, 1994) using centrifuge tests on tunnel models in dry sand. Specifically, Chambon and Corté examined pressures at which face stability was lost and observed the post-instability ground movements associated with tunnel failures at various depths. In these tests, the physical model of the tunnel consisted of a rigid cylinder with a rubber membrane placed at the end of the cylinder to model the tunnel face. Face support was provided during testing through the application of either air or water pressure to the face membrane. Chambon and Corté then examined face stability by decreasing the internally applied face pressure until collapse occurred. Using a linearly variable displacement transducer (LVDT) mounted to the face membrane, Chambon and Corté successfully generated the first physical modelling data of the relationship between face pressure and face displacement for tunnels in sands, and also found that at very large face displacements, the failure mechanism in the ground is very narrow and propagates almost vertically up from the tunnel to the ground surface.

Limit analysis techniques have also been used to examine the face stability of tunnels in sands by Leca and Dormieux (1990). These upper and lower bound solutions predict a range of pressures for which tunnel face instability might occur, of which the upper bound solutions compared well to the reported centrifuge data of Chambon and Corté (1989), while the lower bound solutions yielded a failure pressure significantly higher than that observed in the lab.

However, the unknown stiffness of the tunnel face of Chambon and Corté’s model tunnel (i.e. membrane operated under pressure control) leaves uncertainty as to whether significant stress redistribution due to arching might have occurred in front of the tunnel face during gravity turn-on (or “spinup”) in the centrifuge model. For example, as the horizontal stresses in the soil.
increase during spinup, the tunnel face would deform into the tunnel, unless the internal pressure was increased exactly at the same rate to balance the increased stresses from self weight. Such inward soil displacements would lead to a decrease in horizontal stresses that would deviate from those corresponding to zero lateral strain (i.e. Ko) conditions. Conversely, if the internal pressure in the tunnel was larger than the horizontal stresses from self weight the outward displacements of the tunnel face would lead to a passive soil condition, where there is an increase in the horizontal stresses from the increased strain induced by the tunnel face. Thus, it cannot be ruled out that the low face collapse pressures reported by Chambon and Corté (1994) of a few kilopascals are directly attributable to their adopted boundary condition.

As mentioned at the outset of this chapter, the construction process of tunnels aims to not only avoid the conditions which will lead to catastrophic loss of support at the face, but also to control the deformations associated with the underground excavation.

The physical modelling data of Chambon and Corté (1994) currently provides the best picture of the collapse mechanism of tunnels in sands. Once face stability had been lost, the centrifuge tunnel model was subjected to a post-mortem examination in which the sand was partially saturated and sectioned to reveal the deformations of the soil above the tunnel as witnessed by deformed buried layers of coloured sand. However, this method of assessing the deformation only captures the post-collapse failure mechanism, not the deformations leading up to the point of instability. Indeed, the upper bound failure mechanisms postulated by Leca and Dormieux (1990) indicates that at the point of face instability, the failure mechanism is located in a small zone of ground just in front of the tunnel face and by no means extend to the ground surface. However no
physical data exists to confirm this. Thus, additional research is needed to precisely examine the progression of ground deformations with decreasing face pressure.

However in real world engineering practice, tunnelling through dry (cohesionless) sand is extremely rare. At most sites in coarse-grained soils, portions if not all of the tunnel length can be excavated and constructed within the vadose zone (above the groundwater table), where the coarse-grained soil contains enough moisture to create some amount of apparent cohesion. This generalisation holds especially true for urban areas in which tunnels are likely to be constructed (Martin et al, (2003). However, despite this fact, no physical modelling data exists to describe the progression of ground deformations with loss of tunnel face pressure in unsaturated sands.

2.3 Objectives and scope

The objectives of this Chapter are to:

- Develop a combined strain / load controlled face boundary condition in centrifuge tests that provides a well defined face support in order to define the point which tunnel face instability occurs for various tunnel depths in dry sand;
- Track the face pressure – soil deformation relationship throughout the entire longitudinal cross-section of the tunnel model for various tunnel depths in dry sand, using the digital image correlation technique of GeoPIV (White et al., 2003); and
- Provide the first ever experimental data describing tunnelling induced deformation in unsaturated sand.
2.4 Experimental methodology

2.4.1 Centrifuge model details

The mechanics of tunnel face instability, like most other aspects of soil behaviour (soil liquefaction during seismic loading, soil-structure interaction, transport of groundwater contaminants, etc) are inherently gravity dependant phenomena. In other words, the strength, stiffness, and behaviour of a soil vary dramatically with applied stress (e.g. depth). While large scale models would be able to model these phenomena, they are too difficult and expensive to perform in order to examine the effect of different parameters. However, small scale models (at 1 g) would be less expensive and relatively easy to perform in series, these processes cannot generally represent real-world behaviour.

Geotechnical centrifuge model testing is a research technique which overcomes this limitation by subjecting these small scale model experiments to increased acceleration fields to regain similitude between small scale models and real-world soil behaviour. This means that a 1/N scale model will behave like its full-scale prototype if subjected to a centrifugal acceleration of N times gravity (each soil particle weighs N times its normal weight). That is to say that subjecting a small scale model to centrifugal acceleration creates a stress distribution that starts from zero at the model surface and increases linearly. At depth the soil stresses at homologous points in the small scale model and full scale prototype have a 1:1 correspondence. Therefore centrifuge testing allows for significant benefits over full scale or field testing in terms of cost, time, variability, and experimental control. Further details of centrifuge testing, scaling laws, and sources of error are given by Taylor (1995).
2.4.2 Description of the C-CORE centrifuge

A series of plane strain centrifuge model tests were conducted at the geotechnical centrifuge facilities at the Centre for Cold Oceans Resource Engineering (C-CORE) located in St. John’s, Newfoundland, Canada. Phillips et al. (1994) provide the specific details of the C-CORE centrifuge and its facilities, and only a brief overview is given here. Shown in Figure 2.1, the C-CORE centrifuge is an Acutronic 680-2 centrifuge, which measures 5.5 metres in radius to the swinging platform, and can accommodate a payload of up to 1.1 by 1.4 metres in plan, and up to 1.2 metres in height over the full plan area, with the headroom increasing to 2.1 metres in the centre of the platform. At the maximum rotational speed of 189 rpm, the acceleration at a package at 5 metres radius is approximately 200 g. The centrifuge can carry a maximum payload of 2.2 tonnes to 100 g at 5 metre radius, reducing to 0.65 tonnes at 200 g due to the increased self-weight of the platform.

2.4.2.1 Description of the tunnel model

In order to address the ambiguity of the deformations prior to and following failure, the tunnel was modelled with the Perspex viewing window of the centrifuge strong box forming a vertical plane of symmetry running longitudinally along the tunnel. The tunnel model that was used is shown in Figure 2.2 and Figure 2.3, with a typical schematic drawn in Figure 2.4. All dimensions regarding the tunnel model were measured using digital callipers to the nearest 0.1 mm, and where dimensions exceeded the 150 mm range of the callipers, a steel tape measure was used to the nearest 1 mm. The model consisted of a nominal 4 inch stainless steel hollow tube, 102 mm in diameter and 190 mm long with a wall thickness of 10 mm, which was split along the centre-plane. A 2 mm thick stainless steel plate, 170 mm long, was welded flush with the side of the model that is opposite to the curved side creating an enclosed semi-circle if the model is viewed from the end representing the tunnel face. Also at the end representing the tunnel face, a C-
shaped aluminium retaining ring was screwed onto the tunnel. The retaining ring had an outside
diameter of 102 mm, and an inside diameter of 88 mm. Emery paper was glued on the inside
portion of this retaining ring so that the face piston was firmly positioned at the inside invert of
the tunnel when the balloon is fully inflated during the model making process. This retaining ring
provides a reaction to the tunnel face piston, to initially attain zero lateral face displacement in
advance of the tunnel face. Additional information regarding the tunnel face piston and balloon
are given later in this section.

At the end opposite to the face, a stainless steel vertical box section was welded to allow room for
pressure fittings that would be used to inflate and deflate the balloon. The bottom of this section
was closed off by welding a steel plate to it, so that it was flush with the invert of the curved
tunnel section, and the top was covered with an aluminium plate, and can be bolted down to
prevent sand from migrating into the tunnel model during the tests.

The tunnel face was modelled by using a piece of aluminium that is in the shape of a half-circle,
with a radius of 43 mm and a thickness of 5 mm. A piece of glass cut in the shape of a half-
circle, with radius of 42 mm, was glued onto the outside portion of the aluminium face piston to
help reduce the soil-face interface friction, as seen in Figure 2.6. A piece of emery paper was
then glued onto the outside of the face to fit around the glass. For the unsaturated sand tests, two
layers of geotextile were attached to the back of the tunnel face piston. The geotextile layers
were cut in the half-circle shape, with a radius of 45 mm, to fit the inside diameter of the tunnel.
Also a series of ten 6.35 mm diameter sintered brass porous elements were fitted into the
aluminium face, as shown in Figure 2.6, to ensure a hydraulic connection through the tunnel face
piston. The geotextile or the pore pressure transducer (PPT) brass porous elements were not needed for the dry sand tests.

The pressure fittings for the interior of the tunnel were constructed by connecting one leg of a brass T-junction to a barbed nozzle. A miniature pressure transducer (GE Druck model PDCR-81), that was calibrated and read to an accuracy of 0.1 kPa, was placed inside of the barbed nozzle, with the wire leading out one of the remaining legs of the T-junction, making sure that all gland seals are in place so that no fluid can leak around the PPT wire. A thick rubber balloon was cut to length, with the mouth of the balloon fitted around the nozzle of the bronze fitting. The PPT head was positioned just inside of the nozzle so that it would not make any contact with the balloon. Pressure tubing was attached to the last remaining leg of the T-junction, which was connected to a valve on the strongbox to control fluid flow in and out of the balloon. Two o-rings were then rolled onto the balloon to securely attach it around the nozzle. A galvanized steel bracket was then attached to the bronze fitting, so that the fitting could be firmly attached to the backing plate via two screws. The backing plate served to provide a reaction to the inflated balloon at the end opposite to the tunnel face. Once the backing plate was attached to the clamp and fitting, a small hole was cut into the bottom portion of a plastic bag, so that the balloon would fit through the hole. A third, larger o-ring was placed over the balloon, but inside of the plastic bag, as shown in Figure 2.5b. The whole assembly was then positioned inside of the model tunnel, and secured in place by screwing the backing plate to the tunnel via three screws, two located on the flat side of the tunnel, one located on the curved side of the tunnel. For the dry sand tests, baby powder was then sprinkled lightly into the plastic bag to coat the balloon in an attempt to lubricate the balloon as it inflated inside of the tunnel. However, no baby powder was
used for the unsaturated sand tests. Details of the tunnel model used for the dry sand and unsaturated sand tests are given in Table 2.1.

2.4.2.2 Description of the strongbox setup
C-CORE’s plane-strain strongbox was used for all centrifuge tests. The box has internal dimensions of 900 mm by 300 mm in plan, and is 400 mm deep as measured to the nearest 1 mm using a steel tape measure. A picture of the strongbox is given in Figure 2.7a and Figure 2.7b. At the front face of the strongbox, there is a 76 mm thick window made of clear acrylic. This strongbox allowed for the measurement of ground movement using photogrammetry and particle image velocimetry software, which is further explained in White et al. (2003).

For each test, a geotextile layer was used to cover the bottom of the strongbox in order to protect a series of drainage ports in the bottom from clogging with sand. Next a 12 mm (0.5 inch) thick steel extrusion plate was placed inside the strongbox. This steel plate had tapped holes, which allowed for the secure attachment of the model tunnel to the plate. A 7 mm thick piece of laminated glass, measuring approximately 900 mm by 400 mm, was then secured directly against the inside portion of the strongbox window. This piece of glass served the purpose for ease of GeoPIV control marker placement, and it also helped to save the acrylic strongbox window from being scratched throughout the testing series, which would thus affect the performance of GeoPIV software. A similar piece of glass was also secured to the inside wall that is opposite to the strongbox window to ensure similar boundary conditions on both walls of the strongbox.

The strongbox was then mounted in the centre of a 9.5 mm thick steel mounting plate. This was done for ease of placement and removal of the strongbox from the centrifuge swing. It also allowed for secure placement of the digital photography equipment (Canon Powershot G6 camera
and two lights), and signal conditioning box for measurement devices. Once the model was completely prepared, as described in Section 2.4.2.3 and ready to test in the centrifuge, an aluminium top plate was bolted to the top of the strongbox, in order to reinforce all walls of the strongbox for flight in the centrifuge.

2.4.2.3 Model preparation and testing processes

Model preparation for the dry sand and unsaturated sand tests would begin by constructing the model tunnel, as explained in Section 2.4.2.1, noting that a new balloon and plastic bag would be used for each test. The model tunnel was then secured to the 12 mm thick steel plate in the bottom of the strongbox via two L-shaped brackets, shown in Figure 2.2 and Figure 2.3. The tunnel face piston was also positioned inside the tunnel, in such a manner that if one was to look through the strongbox window at the model tunnel, the face would be positioned flush against the glass window with the top edge of the piston was towards the inside crown of the tunnel. The face piston was temporarily taped to the tunnel so that it would not move from this location upon balloon inflation. The tape was immediately removed after balloon inflation.

Once it was determined that the tunnel and face piston were flush against the window, and completely secured to the steel extrusion plate, the pressure tubing was then attached to a valve on the strongbox wall. This valve was then directly attached to an air pressure regulator-valve system on a portable air supply, shown in Figure 2.7b. The pressure regulator on the portable air supply was then set to a nominal value of 130 kPa, and all valves were slowly opened. Air then filled the balloon, and the tape was removed from the face piston and tunnel, as the tunnel face was then secured in its desired position. A nominal value of 130 kPa was chosen as it is an order of magnitude higher than the expected collapse pressures, and significantly higher than the at-rest horizontal earth pressures which could be expected in the model. In this way, this high initial face
pressure will ensure the face remains at a zero deformation condition during model making and the application of centrifuge acceleration to an in-situ stress state representative of a full-scale tunnel.

Once the tunnel face was locked in place with the high balloon pressure, sand was then pluviated into the strongbox. The C-CORE pluviator system, shown in Figure 2.8, is contained in a separate room from the main laboratory bay, and is comprised of a hopper which is able to translate in a back-and-forth motion on a set of rails, which are at a fixed height on a frame. The strongbox was then placed within this frame, with a series of knife-edges placed around the inside perimeter of the strongbox. The knife-edges helped to ensure an even soil density around the edges of the strongbox, as they keep sand grains from bouncing off the horizontal surfaces of the box. Hopper motion is from a stepper motor which is controlled by a computer located outside of the pluviation room. The computer program that controls the stepper motors is able to be modified to change variables such as hopper speed and travel distance of the hopper.

Soil samples were created by continuously filling the hopper with sand. The hopper moves back and forth along the tracks at a prescribed rate, dumping sand out of the bottom of it, and into the strongbox. The density of the sand is governed by the height of the strongbox under the frame. For the tests described in this chapter, the target relative density of the soil was approximately 80% throughout the entire sample. This was obtained by placing the strongbox on a series of supports that raised the bottom to approximately 450 mm off the pluviation room floor, measured to the nearest 1 mm using a steel tape measure. The density of the sand was measured at various elevations within the strongbox, and it was found that the density of the sand remained largely similar from the bottom to the top of the strongbox. The average measured density and unit
The weight of each test is provided in Table 2.2. More information regarding pluviation can be found in Taylor (1995). Direct shear tests (ASTM D 3080-90) performed at the target relative density of approximately 80% yielded a peak friction angle of 47° and a residual friction angle of 35° (for normal stresses of 28.18 kPa, 57.1 kPa, and 85.27 kPa). The minimum and maximum dry unit weights of the sand used are 12.7 kN/m³ and 15.8 kN/m³, respectively, and the sand has a specific gravity of 2.65.

For the unsaturated sand ground condition tests, the pluviated model then needed to be introduced to water. This was accomplished by attaching a saturation cylinder, filled with deionised water, to the drainage ports in the bottom of the strongbox. A layer of geotextile was placed on the model soil surface, followed by an approximately 50 mm thick layer of coarse sand, then several weights. This surcharge overburden pressure served to decrease saturation time, while helping to prohibit a quick condition of the sand as it was being saturated. Once the sand was fully saturated, the flow of water into the strongbox was stopped, and the soil surcharge was removed from the soil model surface. The aluminium top plate was then secured to the strongbox. The strongbox was then allowed to drain, and the coarse sand allowed to become unsaturated.

The strongbox was then carefully transported to the centrifuge chamber and placed into the centrifuge swing. All data measuring devices were attached to the data acquisition computer, and secured for flight in the centrifuge. The air supply regulator to the balloon was switched from the portable air supply tank to the air supply mounted to the centrifuge swing. This air supply was controlled by an electronic regulator in the centrifuge control room. The centrifuge chamber was then locked. The data logger was then set to record data at 2 Hz, and upon centrifuge spinup, photographs were taken at 20 second intervals and saved to the digital camera’s memory card.
However, once the target acceleration of 50 g was reached, the camera settings were changed to take pictures every 6 seconds, saving the photographs to the control room computer. The testing began by spinning the centrifuge up to the target 50 g acceleration in 10 g steps, as the data recorded. Once the target test acceleration of the centrifuge was attained, the pressure inside the balloon was decreased at a constant rate using the electronic regulator (0.1 v every 30 seconds) until the tunnel face had completely collapsed. The centrifuge was then spun down, all measuring devices turned off, and the strongbox was taken off the centrifuge swing and excavated. Further explanations of centrifuge testing are explained in Taylor (1995).

2.5 Results

2.5.1 Face boundary conditions
The tunnel face boundary condition consists of a rigid plate that is able to translate and/or rotate when the internal tunnel face pressure is reduced. An idealised diagram of the pressures acting on the tunnel face is shown in Figure 2.9a. Prior to any tunnel movement, the vertical effective stress acting on the tunnel starts at zero at the soil surface and increases linearly with depth. The horizontal component of this stress acts on the tunnel face and is assumed to be an at-rest condition. Inside the tunnel, an air pressure distribution acts opposite to the horizontal at-rest stress outside of the tunnel face.

The tests presented in this thesis were performed using a pressure-controlled boundary imposed at the tunnel face. That is to say that the face was supported using either air or water pressure, with displacement of the tunnel face occurring due to the decrease in the internal tunnel pressure acting on it. Movements of the ground were then tracked using GeoPIV, and are associated with the displacement of the tunnel face.
The pressure-controlled boundary is not the ideal condition; imposing a strain-controlled boundary would be however. A boundary that is strain-controlled would consist of the tunnel face moving horizontally inward in discrete increments, operated mechanically. Therefore ground movements measured using GeoPIV could then be associated with a specific inward horizontal tunnel face displacement. This approach was initially abandoned due to the inherently large construction tolerances of such a rugged small-scale tunnel model. These tolerances caused inaccurate measurement of the soil pressure acting on the tunnel face. Yet measurement of the face pressure is a crucial element of the research presented herein, so a pressure-controlled model was used since it allowed for the measurement of the soil pressures.

In spite of being able to measure the soil pressures acting on the tunnel face, the pressure-controlled tunnel face exhibited a “stick-flip” phenomenon as it moved. Namely, as the internal tunnel face pressure was reduced, the bottom of the tunnel face moved inward slightly more than the top of the face. The face would then fall to the tunnel invert, and the bottom would appear to be “stuck” in more or less the same position, as the face would then “flip” into the tunnel. The movement was such that the top of the face would move into the tunnel much more relative to the bottom, giving it the appearance of rotating about the bottom, even though the bottom of the face was not fixed or pinned to anything. This movement was typical for the tests, and is represented as a sketch in Figure 2.9b.

For the dry sand tests, the movement of the tunnel face was largely similar, and are discussed in detail here. Plotted in Figures 2.10 – 2.13, are the locations of the tunnel face for seven different instances (Points O-F, as explained in Section 2.5.2.1), measured by GeoPIV. In these figures the
horizontal deflection scale has been greatly exaggerated to illustrate the detailed initial movements of the tunnel face. Despite these appearances, the inclination of the tunnel face, measured by GeoPIV, is considerably less than 0.5° at the moment of tunnel collapse (Point B) as reported in Table 2.3. For the sake of completeness the horizontal, vertical, and resultant displacements at the mid-elevation, top-elevation, and bottom-elevation of the face for each test are also reported in Table 2.4.

The variability of movement between the top and bottom of the tunnel face causes a problem with relating ground movements with a distinct face displacement. Therefore, it is reasonable to average displacements at the mid-height of the face since this is the region where the variability is minimised. Thus, only the horizontal component to face displacement was examined since the vertical movement is only a small component to the resultant movement. All face displacements here will be reported for an average horizontal face displacement at the mid-elevation of the tunnel face.

Between the start of each test (Point O) and Point A (corresponds to 0.1 mm of horizontal face displacement) very little horizontal displacement occurred for each test, and is difficult to determine with the naked eye. This is consistent with all tests, as depicted by the second to last position of the tunnel face from left hand side of each plot in Figures 2.10-2.13.

From Point A to B (Point B is taken as the onset of face instability), the bottom of the tunnel face generally moved inward more than the top of the face. This was the case for tests Dry-0.5 and Dry-2, where the bottom of the face moved slightly inward, which caused only a vertical downward movement at the top of the face. However, for tests Dry-1 and Dry-1.5, this was not
the case. For test Dry-1, the face stayed at the same position but translated slightly in and down into the tunnel. Likewise for test Dry-1.5, the top of the tunnel face moves slightly inward but the bottom of the face does not (Figure 2.12), and in fact the bottom of the face for this test appears to have been stuck.

As seen in Figures 2.10-2.13, the bottom of the face moves into the tunnel suddenly and immediately falls to the invert of the tunnel. This causes a downward vertical movement of the tunnel face only, and is consistent with tests Dry-0.5, Dry-1, and Dry-2. For test Dry-1.5, the tunnel face goes directly from Point B to Point F.

Beyond Point C, as seen in Figures 2.10-2.13, the bottom of the tunnel face appears to remain in more or less in the same position within the tunnel. Face displacements are then due to the more significant inward movement of the top of the face rather than the bottom. This is the trend that is consistent from Points C to D, D to E, and E to F. This is shown physically in Figures 2.10-2.13.

The area of the balloon acting on the inside of the tunnel face was found to vary between the beginning and end of each test. Therefore, the area of the balloon in contact with the tunnel face is smaller at larger displacements and is due to the balloon’s round geometry when inflated. However, the effect of this is considered to not have much influence at the very import point of tunnelling failure (Point B) since a review of the test photographs show that much of the area in contact with the face at Point O is still in contact with the face at Point B.
2.5.2 Tests in dry sand

2.5.2.1 Ground displacements with decreasing face pressure for a typical test:
Ground deformations with decreasing tunnel face pressure are first examined for Test Dry-1. The horizontal displacement (averaged between 10 mm above and below the mid-elevation of the tunnel face) is plotted against tunnel balloon pressure in Figure 2.14. Seven different points of average horizontal face displacement, at the mid-elevation of the tunnel face, are highlighted on this Figure (Points O-F). Resultant ground displacement contours and vectors, along the longitudinal centreline of the tunnel, are plotted for Points A-F in Figures 2.15 – 2.26. Images taken during the test at the seven values of face displacement are presented in Figures 2.27. All ground movement contours given in the plots in this Chapter are contours of resultant ground movement. The precision of ground deformations is better than 1/10th of a pixel (White et al., 2003). With the resolution and focal length of the camera and lens system used in the present study, this results in measurements of a precision of 0.01 mm. The 0.1 mm contour interval therefore represents the first well-defined contour of ground movement which can be plotted using this system.

Figure 2.14 shows very little (less than 0.1 mm) face displacement as the tunnel face balloon pressure decreases from its initial value (Point O) of 125 kPa to 35 kPa. At approximately 30 kPa, Point A on the Figure 2.14, the first visually noticeable movement of the tunnel face was detected. At this point the tunnel face moves slightly inwards, creating very small movement of the ground immediately adjacent to the tunnel face (Figures 2.15 and 2.16). The extent of ground deformations is localised between the centreline and invert of the tunnel, with the 0.1 mm contour interval extending only approximately 10 mm out from the tunnel face (Figure 2.15).
Beyond Point A the tunnel face progressively moves inwards in very small increments, as the tunnel face balloon pressure is gradually reduced, that renders the slope of the pressure-displacement curve smaller up until Point B (which is the instant before the first sudden horizontal movement of the tunnel face). Ground movement contour plots at Point B (Figure 2.17) show that the ground movement extends further into the ground, with the 0.1 mm contour interval extending from the tunnel invert to the tunnel crown, and extending approximately 25 mm ahead of the face. Ground movements near the face exceed 0.2 mm. Point B is defined as the point of initial instability of the tunnel face, which is also referred to as local failure. Ground movements at and before this point are small and localised around the tunnel face, and any ground movements associated with face displacements after Point B become quite large in comparison. After this point, there are relatively larger increases in face displacement with smaller decreases in tunnel balloon pressure.

Point C on Figure 2.14 corresponds to the instant immediately after the first rapid increase in horizontal face displacement. This larger displacement corresponds to a larger zone of ground displacement in advance of the tunnel face, as shown in Figures 2.19 and 2.20. Figure 2.19 shows that the failure zone of the ground has grown quite considerably from Point B. The ground failure zone has now extended to approximately 50 mm above the tunnel crown and approximately 40 mm away from the tunnel face. Above the tunnel crown, the failure zone also extends approximately 15 mm behind the tunnel face.

Just after Point C, the face displacements do not increase as fast as they did between Points B and C. This abrupt change in horizontal movement of the tunnel face is attributed to a stick and then slip response of the tunnel face. Although the tunnel face is free to move inward with the
decrease in tunnel face pressure, there are points where the movements of the tunnel face may experience greater or less resistance to inward movement depending on its contact with the tunnel. This response is not associated with the ground movements; rather it is due to the nature of the tunnel face boundary condition imposed in these tests.

Further depressurisation of the tunnel face results in additional movement of the tunnel face, until Point D is reached on the tunnel face balloon pressure-displacement curve. Point D represents a point on the pressure-displacement curve just before the extent of ground movements has reached the ground surface. Figures 2.21 and 2.22 show that the zone of failure now extends about 90 mm above the tunnel crown, but it does not quite reach the ground surface. The failure also extends out 50 mm away from the tunnel face and extends approximately 40 mm behind the tunnel face above the tunnel crown.

Beyond Point D on Figure 2.14, any further decrease in the tunnel pressure results in sudden and substantial increases in the shape and extent of the failure mechanism, as it reaches the ground surface causing settlement. The conical-shaped failure zone starts at the tunnel invert and increases linearly outward to the ground surface, extending 75 mm from the tunnel face, and extending back 40 mm behind the tunnel face. Between Points D and E on the pressure-displacement curve, there is such sudden large movement of soil particle within the failure zone, that GeoPIV is unable to track ground movements. Thus movement within the 4 mm contour line represents ground movement that is much greater than 4 mm, and consequently vectors are not shown for this region in Figure 2.24. The maximum vertical settlement at this Point, as examined in Section 2.5.2.4, is approximately 2 mm.
As the tunnel face pressure is decreased even further, sudden large displacements of the tunnel face occur. Point F on Figure 2.14 represents the average horizontal face displacement (at the mid-elevation of the tunnel face) closest to 7 mm. A clearly defined failure mechanism can be seen (Figures 2.25 and 2.26), with much larger surface subsidence compared to the contour plots from Point D.

2.5.2.2 The face pressure-displacement relationship
Plots of face pressure against face displacement are presented for all tests in dry sand tests in Figures 2.28 - 2.31. The shape of the curves is similar throughout the range of C/D values examined, with most exhibiting the same principal features (Points O-F) as discussed in Section 2.5.2.1. The one exception is test Dry-1.5 (dry sand, C/D = 1.5). In this test, the tunnel face went directly from Point B to Point F (Figure 2.30). This test did not demonstrate the stick/slip response of the tunnel face that all other tests exhibited, as presented in Section 2.5.1 and discussed in Section 2.6.1.

The point of interest for the tests presented here is Point B. This point represents the onset of local tunnel face failure, and the beginning of large deformations. The face pressure at this point is therefore not only important to ensure face stability but also to control and limit ground deformations associated with the tunnel construction process. The pressure at Point B for each test is summarised in Table 2.5, which is measured to an accuracy of 0.01 mm using GeoPIV. The pressures associated with Point B are close, ranging from 13.2 kPa to 18.8 kPa. Interestingly however, the displacements of the tunnel face (also presented in Table 2.5) vary with tunnel depth, ranging from 0.08 mm to 0.26 mm. The significance of these results is discussed further in Section 2.6.2.
2.5.2.3 Ground movements with horizontal face displacement

The ground deformations associated with the loss of tunnel face support are presented as face balloon pressure-displacement curves in Figures 2.28 through 2.31, and in snapshots of deformation contours in Figures 2.32 through 2.39 for each of the four burial depths investigated. The horizontal face displacements associated with Point B vary for each test examined and are summarised in Table 2.5. As previously mentioned, Point B is the onset of local failure since ground movements before it are small and localised around the tunnel face; yet after this point ground movements become quite large in comparison, and eventually lead to surface subsidence.

For small horizontal face displacements at Point B (less than 0.26 mm for the range of C/D values examined), ground movements are localised around the tunnel face. At very low cover depths (C/D = 0.5) the zone of ground movement is quite small, starting near the tunnel invert and extending up and out to 15 mm ahead of the face and closing near the tunnel crown (Figure 2.32). For sufficiently deeper tunnels (C/D > 1.0) the zone of ground movement begins near the tunnel invert and extends up and out from that location to 25 mm ahead of the tunnel face, and closes at the tunnel crown (Figures 2.33 through 2.35). Of particular mention, the ground contours associated with test Dry-1 at Point B start at the tunnel invert, where the others start approximately 10 mm above the tunnel invert. This is likely due to the slight differences in the movement of the tunnel face, as described in Section 2.5.2.1. For Dry-1, the tunnel face translated downward and inward into the tunnel from Point A to B, whereas for the rest of the tests examined, the bottom of the tunnel face moved slightly inward. The translation of test Dry-1 possibly created a larger volume for the ground ahead of the face to move into rather than the slight inward movement of the bottom of the face as seen in the other tests.
At larger face displacements (Point F), the zone of ground movement is much larger compared to those at Point B for the dry sand tests, as expected. For all tests, the zone of ground movement at a horizontal face displacement of 7 mm starts at or near the tunnel invert, and angles up and out from that point. Then, at the tunnel crown, the extent of the zone of movement extends approximately 40 mm ahead of the tunnel face. From this point the extent of ground movement reaches the soil surface. The distance that the extent of movement reaches ahead of the tunnel face at the soil surface varies between tests; the smallest reaches 60 mm ahead of the tunnel face (C/D = 0.5) and the largest reaches 100 mm ahead of the tunnel face (C/D = 1.5). The extent of ground movement also reaches behind the tunnel face above the tunnel crown, ranging from 25 mm to 55 mm behind it (Figures 2.36 through 2.39). As described above, the manner in which the tunnel face moved into the tunnel is likely the reason why the ground contours for test Dry-1 starts at the tunnel invert; while for the rest of the tests, the ground contours start approximately 10 mm above the tunnel invert.

In all contour plots at large horizontal face displacement, a zone of larger ground movement can be observed (4 mm contour interval). Movements within this contour interval are greater than 4 mm; however the velocity of the ground instability at this stage of the experiment greatly exceeded the frame rate of the image acquisition (0.17 Hz), with the result that the displacement field could not be defined over this time increment. For depths larger than C/D of 0.5, this zone can be defined as having a bulb shape to it. This bulb follows an angle that the extent of ground movement makes below the tunnel crown, but at the tunnel crown it moves vertically upwards. The most ground movement that occurs from face displacement occurs in this zone as sand flows into the tunnel. It is the ground within this zone that Chambon and Corté (1994) describe as the failure mechanism that develops in the ground due to tunnelling failure. However, any adjacent
sand particles that are outside the bulb-shaped failure mechanism tend to move inwards towards the failure mechanism.

2.5.2.4 Settlements along the Z-axis

Ground surface settlement profiles along the z-axis are shown in Figures 2.40 through 2.43 for all tests in dry sand. Settlement profiles are given for high tunnel face displacement (Point F), as no settlement of the ground surface was measured for relatively small face displacements (Point B) for each test.

For each test, the settlement profile is approximately centred between 15 mm to 20 mm ahead of the tunnel face, varying slightly between tests. However, both the width and the depth of the settlement profiles vary greatly between each test. For test Dry-0.5 (dry sand, C/D = 0.5), a width of 65 mm and a depth of 12 mm was measured, which is similar to test Dry-1.5 (dry sand, C/D = 1.5), where a width of 80 mm and a depth of 15 mm was measured. Conversely, test Dry-1 and Dry-2 (dry sand, C/D = 1.0, and C/D = 2.0 respectively) exhibited smaller settlement profiles. In test Dry-1 a settlement profile with a width of 75 mm and a depth of 2 mm was measured, while a settlement profile with a width of 45 mm and a depth of 2 mm was measured. At such large strain conditions such as this, it is conceivable that the tunnel face has some slight rotation in the third dimension (i.e. rotation about the y-axis) which would result in increased variation in the soil deformations observed at the plane of symmetry (i.e. the plane of measurement). It should be noted that the experiment was designed specifically to investigate the tunnel face displacement at the point of instability (Point B), not the very large strain case of the ground deformations reaching the soil surface. Therefore, it should be expected that the deformations observed at small deformations (Point B) are more consistent than those observed at large strain (Point F).
2.5.3 Tests in unsaturated sand

Plots of face balloon pressure versus face displacement are presented for the tests in unsaturated sand (Figures 2.44 through 2.46). The shape of the curves is similar throughout the range of C/D examined, all exhibiting similar events as discussed in Section 2.5.2.2. As summarised in Table 2.5, the pressures at initial instability of the tunnel face (Point B) are also similar throughout the range of depths examined. However, the displacements of the tunnel face at Point B decreases with increasing tunnel depth.

2.5.3.1 Ground movements at initial instability of tunnel face

Resultant ground movement contour plots for tunnels in unsaturated sand are plotted in Figures 2.47 through 2.49, for Point B. In these contour plots, no movement of the ground is observed for tests Unsat-0.5 and Unsat-1. However, a very small zone of ground movement was seen in test Unsat-2, with the 0.1 mm ground movement contour starting approximately 75 mm above the tunnel invert, extending 10 mm ahead of the tunnel face, and closing just below the tunnel crown.

2.5.3.2 Ground movements at large tunnel face displacements

Resultant ground movement contour plots for tunnels in unsaturated sand are plotted in Figures 2.50 through 2.52, for values of large horizontal face displacement (Point F). In these contour plots, the extent the ground movement begins about 10 mm to 20 mm above the tunnel invert. It then extends upwards at an angle to approximately 20 mm ahead of the tunnel face, which is roughly the elevation of the tunnel crown. From that point, the extent of ground movements extends vertically upwards, until approximately 40 mm below the ground surface where it then extends outwards (50 mm ahead of the tunnel face) and to the soil surface. Movement also occurs behind the tunnel face above the tunnel crown, where the zone of movement extends 15 mm behind the face to the ground surface.
However, for deep tunnels (C/D = 2.0), the zone of ground movements does not reach the soil surface; the zone of ground movement starts near tunnel invert and angles up and ahead of the face by 20 mm. At the elevation of the tunnel crown, it then extends vertically upwards an addition 100 mm, with the zone of movement closing. Stability of the ground above this zone is ensured by arching with forces likely transferred to the tunnel crown. The zone of ground movement then closes at the tunnel crown with no movement behind the tunnel face.

2.5.3.3 Settlements along the Z-axis
Ground surface settlement profiles along the z-axis are given in Figures 2.53 through 2.55 for all tests in unsaturated sand. As seen for the tests in dry sand, no ground displacement was seen at the soil surface for small face displacements (Point B), with only settlements given for relatively larger face displacements (Point F).

For all tests in unsaturated sand, the settlement profile is approximately centred 10 mm ahead of the tunnel face. Again, the widths and depths of the settlement profiles vary with depth of tunnel. For test Unsat-0.5 (unsaturated sand, C/D = 0.5), a settlement profile with a width of 50 mm and depth of 8 mm was measured, while test Unsat-1 (unsaturated sand, C/D = 1.0) had a surface settlement profile with a width of 65 mm and a depth of 12 mm. For test Unsat-2 (unsaturated sand, C/D = 2.0), no surface settlement profile was measured even at very high face displacements (7+ mm).

2.6 Discussion

2.6.1 Influence of the boundary conditions
Boundaries associated with the centrifuge testing discussed here are the stiff boundaries of the strongbox, the tunnel model, and the “semi-rigid” boundary at the tunnel face. The boundaries at
the strongbox and tunnel model are rigid at the stresses relating to centrifuge testing at 50 g. The tunnel face however is materially stiff, but is permitted to move during the test.

According to Lee et al. (1992) and Peck (1969), subsidence due to tunnelling is a function owing to numerous parameters. Tunnels excavated with closed face techniques (such as TBMs) advance in a stop and go fashion as the final lining is placed behind the TBM, where ground movements can occur due to the nature of this process as there is a gap created between the final tunnel lining and the TBM itself, as well as ground movements due to face stability. The tunnel model was designed as a rigid container (10 mm thick stainless steel walls, which remains stiff under increased soil stresses due to centrifuge acceleration). It was also securely attached to the strongbox floor in an effort to reduce differential settling which may occur if the tunnel was supported by soil beneath it. This was done in order to eliminate ground movements due to anything other than tunnel face displacement.

The tunnel face boundary does vary between tests, as mentioned in Section 2.5.1. This variability, however, is not thought to have a great effect on the overall shape and extent of ground movements, nor is it thought to greatly influence face pressures. One example is the difference seen between the extents of the ground movement contours shown in Figures 2.32-2.35. For test Dry-1, the ground movement contours start at the tunnel invert while the ground movement contours start approximately 10 mm above the invert for the other dry sand tests.

The only test that did not exhibit the same stick-flip phenomenon of the tunnel face, as seen by all other tests, was Dry-1.5. As mentioned in Section 2.5.2.12, the tunnel face skipped Points C, D, and E, suddenly going from Point B to Point F. In fact, the tunnel face was seen to not be in
complete contact with the rubber balloon. This implies that the tunnel face must have been caught and held in place, perhaps on a sand grain. So as the pressure inside the rubber balloon decreased, the contact between the balloon and the face decreased, causing the internal pressure on the tunnel face to decrease to the extent where the horizontal pressures acting on the outside of the tunnel face were much greater. This likely caused the immediate sudden inward movement of the tunnel face, which happened to correspond with 7 mm of horizontal face movement at the mid-elevation of the tunnel face.

For test Dry-1.5, the resultant ground movement contours are considered valid since the face displacements between Point O and Point B are relatively small (less than 0.26 mm), which create the local ground movement contour patterns exhibited by all tests. Also, the pressures associated with Point B are considered valid since the pressure reported in Table 2.5 is in accordance with the range of pressure examined in the other tests that saw the stick-flip phenomenon of the tunnel face. The ground movement contours at Point F, and its associated face pressure are also similar to those tests that saw the stick-flip tunnel face phenomenon.

While there is slight variability in the exact location of the tunnel face at each Point along the face pressure-displacement curve between tests, the way the face moves is largely similar, and creates similar ground movement patterns (Figures 2.32-2.39) and similar face pressures (Table 2.5). Therefore it can be concluded that the face does not affect the resulting conditions, given that the ground movements in all tests were similar in both shape and extent. The only difference is whether the zone of ground movement starts at, or just slightly the tunnel invert which is affected by tunnel face boundary conditions. However, the overall shape and extent of the ground
movement is still very similar, so the fashion in which the tunnel face moved is deemed to not have an influence on the results.

Though the ground movements and face pressures measured in these experiments most likely will not correspond to field conditions, the boundary conditions are well defined. The results can be used by an analyst to calibrate a finite element model to predict ground movements due to tunnel face displacements. The test procedures used for each experiment permits comparing the results of each test to determine the influence of various components associated with tunnelling (i.e. depth and ground type).

**2.6.2 Influence of tunnel depth**

At very low face displacement (Point B), there is little effect of tunnel depth on shape and extent of ground movements due to horizontal displacement of the tunnel face for tunnels in dry sand. The ground movements associated with Point B are very similar for all C/D values examined for tunnels in dry sand, and can be described as a local failure mechanism, as movements are localised at the tunnel face, and only extend a maximum of 20 mm ahead of it. At higher horizontal face displacements (Point F), the ground movements exhibit similar shapes for the range of tunnel depths examined (Figures 2.32 through 2.39).

Tunnel depth does not seem to affect the pressure that the initial instability of the face occurs (Point B), as summarised in Table 2.5. However, there may be some correlation between the tunnel face displacement at the onset of tunnel face instability and the tunnel depth. The relationship between face displacement at the point of local failure (Point B) and tunnel burial depth (expressed as C/D) are plotted in Figure 2.56. The results from all four dry sand and all three unsaturated sand tests are presented. Also presented in the figure is a data point
representing the average measured face displacement from tests presented in Chambon and Corté (1991, 1994) at similar gravity levels and sand densities to the test results presented in this chapter. Although there is scatter in the data, both the dry sand and unsaturated sand tests had similar face displacements at the onset of tunnel face instability, ranging from 0.08 mm to 0.26 mm. Interestingly though, is the average displacement for Chambon and Corté’s data is almost double that for the largest face displacement (test Dry-1). Chambon and Corté (1994) state that for the tests they performed, final failure occurred when the face moved about 0.5 mm, and noted that the amount of face displacement did not seem to be affected much by the density of the sand. For the tests presented in this chapter, there exists a general trend of tunnel face displacement at failure (Point B) decreasing as the tunnel depth increases for both dry and unsaturated sand tests. This is most likely a result of higher stress levels in the ground for deeper tunnels, which are recognized to increase the stiffness of a soil. Therefore, the horizontal earth pressure acting on the face increases as the tunnel depth increases, which results in increased soil stiffness. So it is rational that the ground displacement into the tunnel at failure will decrease as a result of the stiffer soil. However, the one anomaly is test Dry-0.5, where the face displacement was less than what was seen in tests Dry-1 and Dry-1.5. The reason for this inconsistent finding is unknown.

2.6.3 Dry sand versus unsaturated sand

At low horizontal face displacements the ground movements in the dry sand tests are much greater in comparison with the ground movements seen in the unsaturated sand, as only little to no ground movements (0.1 mm and less) were seen in the unsaturated tests.

There is also quite a difference between the two soil conditions for larger horizontal face displacements. By examining the contour plots of both dry and unsaturated sand, it can be observed that the zone of bulb-shaped ground movements in the unsaturated sand tests are much
thinner and steeper than those in the dry sand tests, and do not exhibit any funnelling of adjacent ground outside of the failure mechanism due mainly to the apparent cohesion. For example, Figure 2.37 (dry sand, C/D = 1.0) has a much broader zone of ground movement when compared to the same test in unsaturated sand (Figure 2.51). In fact, the extent of ground movement does not reach the soil surface for C/D of 2.0, while it does in dry sand at the same depth.

When the failure mechanism does reach the surface, further differences are also seen between the two soil conditions. For shallow tunnels (C/D = 0.5 and 1.0), the width of settlement profile is thinner (Figures 2.40 – 2.43 and 2.53 – 2.55), as expected given the extent of the resultant ground movement contours for dry sand unsaturated sand tests. Yet for deeper tunnels (C/D = 2.0), no settlement profile was measured in unsaturated sand, while a settlement profile with a width of 45 mm and a depth of 2 mm was measured for the dry sand tests at the same tunnel depth.

The differences between the dry sand and the unsaturated sand tests are likely due to the matric suctions developed in unsaturated granular soils, which give small temporary cohesion to the soil matrix. The total overburden stresses in both the dry sand and unsaturated sand tests are the same; however the pore water pressures in the dry sand tests are zero, while they are negative for the unsaturated tests. Therefore, the effective stresses, and thereby the strength of the unsaturated sand tests will be higher than those in dry sand. The apparent cohesion in the unsaturated sand tests increases the shear strength envelope over the sand exact sand in completely dry conditions (ie. same friction angle).

Based on field observations, Peck (1969) reported the loss of ground for tunnels in various soil conditions, especially dry sand. He stated that for tunnels in dry sand, particularly if the material
is loose, large surface subsidences can develop due to tunnelling. This is especially true if no protection of the top, sides, and face of the excavation are provided, such as forepolling or injection of grout into the soil. He also observed tunnels bored above the groundwater table. These tunnels were successfully bored regardless the method of tunnelling. It was mentioned however, that for those tunnels bored below the groundwater table (fully saturated, dense sands), it is appropriate to extract water from the tunnel region to improve stability. Doing so decreases the pore water pressure which increases the effective stress, and increases stability of the soil.

The matric suctions associated with unsaturated sands (that are relatively dense) are temporarily helpful in helping to maintain face stability while tunnelling. At the onset of ground instability due to loss of internal tunnel face pressure, very little (as seen only in test Unsat-2) to no ground movements occur, especially when compared with tests in dry sand. Even at much higher horizontal face displacements, the matric suctions allow for a much smaller surface subsidence, and for deep tunnels (C/D = 2.0), actually prohibit the failure mechanism due to tunnelling from reaching the soil surface to create subsidence.

2.6.4 Comparison with previous tests
Previous physical tests examining tunnels in sand were undertaken by Chambon and Corté (1994), who examined failure pressures for tunnels of various C/D ratios in dry sand. They state that collapse of the tunnel face occurs in three distinct stages, and their typical results are reproduced in Figure 2.57. They state that the first stage begins at an initial tunnel pressure equal to the active earth pressure at the tunnel centreline. Here the face remains stable (with no measured movement) as the tunnel pressure is reduced, until low pressures are reached compared to the initial pressure. The second stage, referred to as the start of collapse, occurs at a characteristic pressure which they called $p_c$. Any further decrease in internal face pressure from
$p_c$ results in an increment of face movement. They state that holding or increasing the face pressure stabilises the face, which they claim to do to prevent the progression of ground failure. In their final stage, they declare that actual failure occurs, which corresponds to approximately 0.5 mm of face displacement for their tests.

Chambon and Corté (1994) describe tunnelling failure as sudden, and it takes the form of free-flow of sand particles into the tunnel. They also state that if no instantaneous increase in tunnel pressure occurs, a collapse chimney develops consequently resulting in ground surface subsidence. They determine the collapse pressure, $p_f$, from the shape of the face pressure – displacement curve (Figure 2.57), which corresponds to Point B on the tests reported here.

A summary of failure pressures are given in Table 2.6, for the tests performed at LCPC (Chambon and Corté, 1994) and the tests reported here. As seen in the table, the pressures at failure for the LCPC tests are much lower than the pressures observed in the dry sand tests reported here. This discrepancy is postulated to be due to the initial conditions of the LCPC model.

Chambon and Corté report that they initially started with an active earth pressure for the tunnel face pressure. Therefore it cannot be ruled out that upon centrifuge spinup, the LCPC model went from an at-rest soil condition to an active soil condition, which implies inward movement of the tunnel face. If this is true, then ground movements occurred in the LCPC model before the tunnel face pressure was reduced from its initial pressure, which leads to soil arching and a decrease in horizontal effective stress. Thus, since arching likely occurred, the soil pressures acting on the tunnel face initially are due to the soil within the zone of ground movements (due to
face displacement) only, and not due to the entire overburden above the tunnel face mid-
elevation.

To illustrate that the methods employed in this thesis were successful in obtaining an initial zero face displacement condition, vectors of resultant ground movement due to centrifuge spinup are plotted in Figure 2.58, for test Dry-1. As seen by the vectors in the plot, there was only vertical movement of ground due to self-weight compression as the centrifuge increased acceleration from 1 g at rest to the target 50 g for the test. This downward movement is due to the increased self weight of the sand, and is a normal phenomenon experienced in centrifuge testing (Taylor, 1995). No horizontal movement occurred near the tunnel face, as the tunnel face did not move during spinup. Thus, the tests reported here were successful in creating initial boundary conditions that correspond to at-rest soil conditions upon centrifuge spinup. Consequently, the tests reported in this thesis are believed to result in larger failure pressures.

Chambon and Corté present soil failure as narrow collapse “chimneys”. They show post-test sketches of their failure mechanism with inferred lines of shear due to tunnelling failure, based upon their method of determining ground movement. The failure mechanism sketches were obtained by initially wetting the sand, and cutting it along vertical planes which dissect the failure mechanism. The failure mechanism was then traced onto paper. The LCPC tests only show inferred zones of ground movement at very large displacements of the tunnel face, though they only associate those ground movements with the failure pressure. However, dry sand tests reported here show localised ground displacements with horizontal face displacements associated with the failure pressure. Despite having only post-test ground deformations, the LCPC approach was beneficial in showing the shape and extent of ground movement due to very large
displacements of the tunnel face, especially considering that they did not have a viewing window imposed down the centreline of their experiments.

Despite this, the LCPC results can be compared to the dry sand test results reported here at the higher face displacements. For the range of tunnel depths examined, the tests reported here see a much broader zone of ground movement than the LCPC tests do. This is likely due to the fact that the ground movement for the LCPC tests were examined with the human eye, while the ground movement in the tests reported here was measured using GeoPIV. Here, due to the depth of field, GeoPIV precisely measures ground movements to 0.01 mm, whereas the LCPC data can only effectively measure what the human eye can see. As a side note, more ground displacement data is available to those who wish to use it to develop a numerical model to calculate these ground displacements, by contacting the author’s supervisors at the Department of Civil Engineering at Queen’s University (Kingston, Ontario, Canada).

Given the shape of the ground failure patterns described by the LCPC tests, the tests reported here simulate their results well. Also, given that the LCPC tests do not provide initial at-rest soil pressures, the dry sand tests reported here represent a more realistic representation of tunnel face pressures which can be more easily compared to by analytical models.

2.6.5 Comparison with analytical models
Leca and Dormieux (1990) developed upper and lower bound plasticity solutions for local failure of a three-dimensional tunnel heading in frictional soils to predict failure geometry and to predict a range of pressures for which failure may occur. They characterized soil strength with a Mohr-Coulomb yield criteria, with cohesion intercept $c'$, and angle of internal friction $\phi'$. They found that their upper bound solution was in good agreement with the centrifuge tests reported by
Chambon and Corté (1991), which are very similar to those presented Chambon and Corté (1994), and were also shown to be in excellent agreement with the dry sand tests reported here.

Figure 2.59 shows the upper bound solution with the observed extent of ground movement at failure (Point B) for the dry sand tests reported here. The critical geometry for the upper bound solution, given by the dashed lines, coincides reasonably well for the observed extent of ground failure for the dry. However, the upper bound solution overestimates the extent of ground movement for the unsaturated sand test, as no ground movement occurred. However, of particular interest is that the failure pressures for the upper bound analytical solution was in accordance with the pressures that Chambon and Corté reported (Leca and Dormieux, 1990). This led them to conclude that the upper bound solutions are closer to the actual pressures at failure than the lower bound values, and can provide a reasonable estimate of critical tunnel face pressures.

However, the dry sand tests reported here show that the upper bound solutions do not provide conservative estimates of the critical tunnel face pressures. Table 2.7 summarises the failure pressures estimated from Leca and Dormieux’s equation:

\[ \sigma_T = N_S \sigma_S + N_\gamma \gamma D \]

where \( N_S \) and \( N_\gamma \) are weighting coefficients determined from Leca and Dormieux (1990), \( \sigma_T \) is the tunnel face pressure at failure, \( \sigma_S \) is a surcharge pressure, \( \gamma \) is the unit weight of the soil, and \( D \) is the diameter of the tunnel. The measured pressures at failure are seen to be well bound by the
predicted analytical solutions. Therefore, a range of pressures are needed to determine a safe face pressure at which ground movements are kept to a minimum.

Of special notice, however, are the pressures predicted for the unsaturated sand tests (Table 2.7). These pressures are higher than what was expected. It would be expected that, due to the apparent cohesion of the unsaturated sand, the pressure at instability would be less than for the dry sands. This is because the apparent cohesion in an unsaturated sand would increase the shear strength envelope for the same friction angle. However, during the saturation process outlined at the outset of this chapter, a surcharge was added to the soil surface to speed up the time require for model saturation while minimising the risk for a quick condition. This surcharge, though temporary, is likely the reason for the slighter higher densities reported for the unsaturated sand tests than the equivalent dry sand tests (Table 2.2). This then results in the higher pressures reported in Table 2.7.

2.7 Summary and Conclusions

Experimental techniques and procedures were developed to analyse the progression of ground movements with decreasing tunnel face balloon pressure. Four tests were performed at various cover to depth ratios (C/D of 0.5, 1.0, 1.5 and 2.0) in dry sand with a target relative density of approximately 80% and three tests were performed at various depths (C/D of 0.5, 1.0, and 2.0) in unsaturated sand at a relative density of approximately 80%. The dry sand test results were then compared to the physical results of Chambon and Corté (1994) and the analytical models of Leca and Dormieux (1990), while the unsaturated sand tests were compared to the dry sand test results isolate the effect of apparent cohesion in unsaturated sand.
For the tests presented in this chapter, a pressure-controlled face boundary was imposed at the tunnel face to accurately measure the soil pressures acting on the tunnel face. The pressure-controlled tunnel face exhibited a stick-flip phenomenon as it was allowed to displace into the tunnel, with some degree of variability in the movement between each test. The movement of the tunnel face was such that, as the face pressure was reduced, the top of the face would move inwards more relative to the bottom of the face. The entire face would then fall to the invert of the tunnel, were the top of the face would then start to rotate relative to the bottom of the face. As a result of this movement, to correlate a zone of ground movement measured using GeoPIV software with a tunnel face displacement, the horizontal component of face displacement was averaged at the mid-elevation of the face since this is the point of the face where the variability in relative (top versus bottom movement) movement was minimal. This displacement was then correlated to face pressure and resultant ground movement.

For each dry sand test, the relationship between face balloon pressure and average horizontal face displacement at the mid-elevation of the tunnel face was examined. From this relationship, the instant before tunnel face instability (Point B) was determined for each test. In dry sand, it was determined that there was no influence of tunnel depth on the face pressure at this point. Also when the resultant ground movement contours were plotted, it was found that ground movements were small and localised around the tunnel face at this instant. In fact ground movements before this point were small and localised around the tunnel face, but ground movements after this point occurred more rapidly and became quite large and eventually reached the ground surface causing surface subsidence. Therefore this point was defined as the point of tunnel failure. Since the ground movements at the point of face instability were all similar in shape and extent, it was
deemed that the fashion in which the tunnel face displaced into the tunnel had no influence on the results presented here.

For small face displacements the ground movements were small and localised around the tunnel face, regardless of tunnel depth. However, for large tunnel face displacements the zone of ground movements had a bulb shape failure mechanism that reached the ground surface, where sand within the failure mechanism flowed into the tunnel. Due to the lack of cohesion in the dry sand tests, there was movement of ground outside of the failure mechanism that moved in towards the tunnel.

Likewise, the effect of unsaturated sand conditions was not found to have an effect on the pressures at which initial instability of the tunnel face occurred, but rather it was found to have an influence on the extent of the zone of ground movement. The apparent cohesion in the unsaturated sand tests created a thinner zone of ground movement a head of the tunnel face in each of the tests examined when compared to the results of the dry sand tests. In the case of deeper tunnels (C/D=2.0), the zone of ground movement didn’t even reach the ground surface. This is especially important in the field because for the smaller ground movement that occurs, the smaller the volume of ground loss on the surface in the event of tunnel face collapse, which limits potential damage to surface structures and utilities to a smaller zone. For deeper tunnels, this could also provide added safety for surface structures and utilities in such an event.

Chambon and Corté (1989, 1991, 1994), who performed similar tests in dry sand, found that tunnel face pressure at the moment of tunnel face instability to be in the range of 3.5 to 4.0 kPa for C/D ratios from 0.5 to 2.0. They also presented post-instability ground failures that showed
ground movements to be bulb-shaped and reached the ground surface, creating surface subsidence. These ground movements were due to the complete collapse of their tunnel face boundary.

Leca and Dormieux (1990) compared their analysis to results from Chambon and Corté (1989) results and found that their upper bound solutions compared well to the reported centrifuge data of Chambon and Corté, while the lower bound solutions yielded a failure pressure significantly higher. They also predicted a geometry which only coincided well with Chambon and Corté’s horizontal extent of ground movement in front of the face; however it only vertically extended to approximately the tunnel crown, which is not what was found by Chambon and Corté’s results.

The tunnel face pressures at initial instability reported by Chambon and Corté (1989, 1991, 1994) were significantly less than the pressures reported here. The difference is likely due to the initial tunnel face conditions: Chambon and Corté’s tests started at the active earth pressure, which suggests that movement of the tunnel face has occurred during centrifuge spinup. This means that soil arching (soil stress redistribution) could have occurred creating a smaller earth pressure acting on the tunnel face. This was not found to be the case with the tests reported here. Also, the pressures at initial instability, for the tests reported here, were found to be in between the upper and lower bound pressures predicted by Leca and Dormieux (1990).

Also, the ground movements due to tunnel face pressure loss were recorded for six instances for the tests reported here (Points A-F). The ground movements associated with initial instability with the tunnel face were found to be localised around the tunnel face, and coincided well with the zone of ground movement predicted by Leca and Dormieux (1990). However, these
movements were not similar to those reported by Chambon and Corté (1994). Rather, Chambon and Corté’s post-instability ground movements correspond well with the points of higher tunnel face displacement (Points E and F), where the zone of ground movement reached the ground surface.

The results reported in this chapter are applicable only for the specific conditions (i.e. model materials, tunnel depth, sand backfill, geometry, and boundary conditions) examined in the experiments. The results presented here may be used by other researchers to calibrate numerical analyses which can help to better quantify the progression of ground displacements with loss of tunnel face stability in a range of field conditions.
<table>
<thead>
<tr>
<th>Test Type</th>
<th>Test Name</th>
<th>C/D Ratio</th>
<th>Test Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry sand</td>
<td>Dry-2</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dry-1.5</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dry-1</td>
<td>1.0</td>
<td>Rubber balloon filled with air, tunnel face does not have geotextile or porous</td>
</tr>
<tr>
<td></td>
<td>Dry-0.5</td>
<td>0.5</td>
<td>elements, talcum powder for used lubrication</td>
</tr>
<tr>
<td>Unsaturated sand</td>
<td>Unsat-2</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Unsat-1</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Unsat-0.5</td>
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Table 2.1: Summary of tests performed at 50 g. Tests performed at various cover to tunnel diameter ratios (C/D).

<table>
<thead>
<tr>
<th>Test</th>
<th>Avg bulk ρ (kg/m³)</th>
<th>Bulk γ (kN/m³)</th>
<th>Calculated Dry γ (kN/m³)</th>
<th>Moisture content at tunnel face centreline (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry-0.5</td>
<td>1551.8</td>
<td>15.2</td>
<td>15.2</td>
<td>0.0</td>
</tr>
<tr>
<td>Dry-1</td>
<td>1521.9</td>
<td>14.9</td>
<td>14.9</td>
<td>0.0</td>
</tr>
<tr>
<td>Dry-1.5</td>
<td>1571.2</td>
<td>15.4</td>
<td>15.4</td>
<td>0.0</td>
</tr>
<tr>
<td>Dry-2</td>
<td>1503.0</td>
<td>14.8</td>
<td>14.8</td>
<td>0.0</td>
</tr>
<tr>
<td>Unsat-0.5</td>
<td>1580.0</td>
<td>15.5</td>
<td>15.2</td>
<td>1.9</td>
</tr>
<tr>
<td>Unsat-1</td>
<td>1590.2</td>
<td>15.6</td>
<td>15.3</td>
<td>2.2</td>
</tr>
<tr>
<td>Unsat-2</td>
<td>1600.4</td>
<td>15.7</td>
<td>15.3</td>
<td>3.2</td>
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</table>

Table 2.2: Average bulk and dry densities, moisture contents, and calculated unit weights for tests examined.
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<thead>
<tr>
<th>Point</th>
<th>Inclination (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dry-0.5</td>
</tr>
<tr>
<td>O</td>
<td>0.000</td>
</tr>
<tr>
<td>A</td>
<td>-0.067</td>
</tr>
<tr>
<td>B</td>
<td>-0.135</td>
</tr>
<tr>
<td>C</td>
<td>-0.721</td>
</tr>
<tr>
<td>D</td>
<td>-0.631</td>
</tr>
<tr>
<td>E</td>
<td>-0.302</td>
</tr>
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</table>

Table 2.3: Summary of regression line inclination angles for dry sand tests.
<table>
<thead>
<tr>
<th>Point</th>
<th>Mid-elevation</th>
<th>Top-elevation</th>
<th>Bottom-elevation</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Horizontal</td>
<td>Vertical</td>
<td>Resultant</td>
</tr>
<tr>
<td></td>
<td>Dry-0.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>0.10</td>
<td>-0.08</td>
<td>0.13</td>
</tr>
<tr>
<td>B</td>
<td>0.14</td>
<td>-0.15</td>
<td>0.21</td>
</tr>
<tr>
<td>C</td>
<td>0.85</td>
<td>-1.27</td>
<td>1.53</td>
</tr>
<tr>
<td>D</td>
<td>0.92</td>
<td>-1.31</td>
<td>1.60</td>
</tr>
<tr>
<td>E</td>
<td>1.22</td>
<td>-1.62</td>
<td>2.03</td>
</tr>
<tr>
<td></td>
<td>Dry-1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>0.10</td>
<td>-0.06</td>
<td>0.12</td>
</tr>
<tr>
<td>B</td>
<td>0.26</td>
<td>-0.33</td>
<td>0.42</td>
</tr>
<tr>
<td>C</td>
<td>0.90</td>
<td>-1.67</td>
<td>1.90</td>
</tr>
<tr>
<td>D</td>
<td>0.96</td>
<td>-1.70</td>
<td>1.95</td>
</tr>
<tr>
<td>E</td>
<td>1.58</td>
<td>-1.70</td>
<td>2.33</td>
</tr>
<tr>
<td>F</td>
<td>7.80</td>
<td>-1.94</td>
<td>8.09</td>
</tr>
<tr>
<td></td>
<td>Dry-1.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>0.10</td>
<td>-0.06</td>
<td>0.12</td>
</tr>
<tr>
<td>B</td>
<td>0.23</td>
<td>-0.18</td>
<td>0.29</td>
</tr>
<tr>
<td>F</td>
<td>7.28</td>
<td>-6.76</td>
<td>12.68</td>
</tr>
<tr>
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<td>Dry-2</td>
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<td></td>
</tr>
<tr>
<td>B</td>
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<td>-0.09</td>
<td>0.11</td>
</tr>
<tr>
<td>C</td>
<td>1.51</td>
<td>-2.70</td>
<td>3.09</td>
</tr>
<tr>
<td>D</td>
<td>1.64</td>
<td>-2.94</td>
<td>3.37</td>
</tr>
<tr>
<td>E</td>
<td>1.74</td>
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<tr>
<td>F</td>
<td>4.10</td>
<td>-6.53</td>
<td>7.37</td>
</tr>
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Table 2.4: Horizontal, vertical, and resultant displacements (in mm) of the tunnel face at six point (A-F) for dry sand tests.
<table>
<thead>
<tr>
<th>Test</th>
<th>Face balloon pressures (kPa) at values of face displacement</th>
<th>Horizontal face displacements at Point B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Point B</td>
<td>7.0 mm</td>
</tr>
<tr>
<td>Dry-0.5</td>
<td>18.8</td>
<td>14.6</td>
</tr>
<tr>
<td>Dry-1</td>
<td>13.2</td>
<td>8.7</td>
</tr>
<tr>
<td>Dry-1.5</td>
<td>16.2</td>
<td>16.1</td>
</tr>
<tr>
<td>Dry-2</td>
<td>18.1</td>
<td>13.5</td>
</tr>
<tr>
<td>Unsat-0.5</td>
<td>19.5</td>
<td>10.9</td>
</tr>
<tr>
<td>Unsat-1</td>
<td>15.5</td>
<td>9.1</td>
</tr>
<tr>
<td>Unsat-2</td>
<td>14.0</td>
<td>16.8</td>
</tr>
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</table>

Table 2.5: Summary of tunnel face balloon pressures at known values of horizontal face displacement for tests in dry sand and unsaturated sand

<table>
<thead>
<tr>
<th>C/D</th>
<th>Pressure at Point B (kPa)</th>
<th>LCPC failure pressure (kPa)</th>
<th>Queen's Unit weight (kN/m³)</th>
<th>LCPC Unit weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>18.8</td>
<td>3.6</td>
<td>15.2</td>
<td>16.0</td>
</tr>
<tr>
<td>1.0</td>
<td>13.2</td>
<td>3.5</td>
<td>14.9</td>
<td>16.1</td>
</tr>
<tr>
<td>1.5</td>
<td>16.2</td>
<td>-</td>
<td>15.4</td>
<td>-</td>
</tr>
<tr>
<td>2.0</td>
<td>18.1</td>
<td>4.0</td>
<td>14.7</td>
<td>16.1</td>
</tr>
</tbody>
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Table 2.6: Summary of failure pressures for tunnels in dry sand, including results from Chambon and Corté (1994).
<table>
<thead>
<tr>
<th>C/D</th>
<th>Pressures at Point B (kPa)</th>
<th>Predicted Analytical Pressures (kPa)</th>
<th>Lower Bound</th>
<th>Upper Bound</th>
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<tbody>
<tr>
<td>Dry-0.5</td>
<td>18.0</td>
<td>19.0</td>
<td>3.8</td>
<td></td>
</tr>
<tr>
<td>Dry-1</td>
<td>13.2</td>
<td>26.8</td>
<td>3.7</td>
<td></td>
</tr>
<tr>
<td>Dry-1.5</td>
<td>16.2</td>
<td>34.7</td>
<td>3.9</td>
<td></td>
</tr>
<tr>
<td>Dry-2</td>
<td>18.1</td>
<td>36.8</td>
<td>3.7</td>
<td></td>
</tr>
<tr>
<td>Unsat-0.5</td>
<td>19.5</td>
<td>19.6</td>
<td>3.8</td>
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</tr>
<tr>
<td>Unsat-1</td>
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<td></td>
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<tr>
<td>Unsat-2</td>
<td>14.0</td>
<td>38.3</td>
<td>3.8</td>
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Figure 2.19: Contours of resultant ground movement (in mm) for face displacement at Point C, for test Dry-1.
Figure 2.20: Vectors of resultant ground movement for face displacement at Point C, for test Dry-1.
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Chapter 3
Ground Response when Tunnelling in Sand beneath Clay

3.1 Introduction

In urban environments, there has been an increase in the development of buried infrastructure as of late. Thus, there has been an increase in the amount of tunnelling projects for modern transportation and services (e.g., water mains, gas pipelines and telecommunication and electric power networks). However, the construction of tunnels causes unavoidable deformations in the ground. Limiting the magnitude of these displacements is important.

Today, tunnelling in soft ground is mainly carried out by mechanized tunnel boring machines (TBM), which cut and remove ground in front it, while applying a positive face pressure, so as to minimise ground movement into the tunnel. Any loss of pressure will cause ground movement, which is problematic in urban environments.

In theory, if the TBM could perfectly balance the earth pressure acting on the face of the excavation, no deformation should occur at the face. However, the higher the face pressure, the slower and more expensive the construction becomes. Experience has shown that reduced face pressures can be successfully used to construct tunnels in sands without excessive ground deformations (e.g. Peck, 1969, Mair and Taylor, 1997). However the ground movements with these reduced face pressures is largely unknown. Thus, accurate predictions of tunnelling-induced ground movements associated with differing face pressures are essential for an efficient construction process that safeguards adjacent structures and utilities from damage.
Much physical research has been performed to investigate the stability of tunnels in both clays and sands. Mair (1979) has performed centrifuge modelling research to examine tunnel collapse in soft clay. He found that the failure mechanism propagation in soft clay is observed to propagate upwards and outwards from the tunnel invert by several times the tunnel diameter. This creates a very broad settlement trough on the ground surface, and has been further confirmed by physical research by Grant and Taylor (2000) for tunnels in moderately stiffer clays, where surface settlement volume losses are given per decrease in tunnel support pressure.

Likewise, Chambon and Corté (1989, 1991, and 1994) examined the relationship between face pressure and face stability experimentally by using centrifuge tests on tunnel models buried in dry sand. Specifically, Chambon and Corté examined pressures at which face stability was lost and observed the post-instability ground movements associated with tunnel failures at various depths. Chambon and Corté found that the zone of ground movement in dry sands is bulb-shaped and propagates nearly vertically upwards less than one tunnel diameter from the tunnel face, creating a small zone of settlement on the ground surface.

However, despite the centrifuge research performed on tunnels in either clay or sand, very little attention has been paid to tunnels in layered ground, which is a common tunnelling medium. Grant and Taylor (1996) and Hagiwara et al (2000) have examined the case of tunnelling in clay with an overlying layer of sand. They found that the stiffness of the sand layer has a significant influence on the distribution of movements in the clay layer. It was also discovered that the subsurface settlement trough (at the clay-sand interface) is quite underestimated by current design method for determining surface settlement. Currently, no physical modelling data exists which
describes the ground deformations to be expected when tunnelling in sand overlain by clay, which is a fairly common ground condition encountered during tunnelling (Martin et al, 2003).

In this Chapter, an extensive study of the progression of ground deformations with decreasing tunnel face pressure is reported for saturated sand, in which a combined strain / load-controlled tunnel face boundary condition is used to provide a well defined face support, the digital image deformation measurement technique of GeoPIV (White et. al., 2003) is used to track the face pressure – ground deformation relationship throughout the entire longitudinal cross-section of the tunnel model. Tests in saturated sand are used to isolate the effect of sand saturation, and then data is presented to provide the first ever experimental data describing the tunnelling deformation for this specific mixed ground condition (clay over sand).

3.2 Experimental Details

3.2.1 Tunnel model and strongbox
Centrifuge model tests were performed to examine the ground response during tunnelling in a saturated sand overlain by a layer of clay. The decision to model the sand layer in a saturated state is an important one, as a clay layer overlying a dry sand would develop large matric suctions leading to unrealistic values of strength and stiffness of this surficial layer. To avoid this possibility, a groundwater table was modelled in all clay tests at the ground surface. Thus, before examining the effect of the clay layer, tests must be performed with saturated sand tunnelling conditions to examine the effect of saturation on the collapse pressure observed at the tunnel face. Similarly to the tests performed on dry sand discussed in Chapter 2 of this dissertation, these tests were preformed at the C-CORE centrifuge modelling facility in St. John’s NL. Details of the tunnel model for each test are given in Table 3.1.
For this second series of centrifuge tests, the tunnel model was slightly modified from that described in Section 2.4.2.1 of this dissertation, to adapt the tunnel face for saturated ground conditions. In particular, the tunnel face had a series of ten 6.35mm diameter sintered-brass porous elements fitted into it. The purpose of these filter elements is to hydraulically connect both sides of the aluminium tunnel face to ensure that the pressure of the sand’s groundwater pore fluid acts on both sides of the tunnel face. As before, the 43mm radius half-circle tunnel face had a 42 mm radius half-circle glass face affixed to the side that represents the actual tunnel face, with emery paper glued onto the outside of the face to fit around the glass. Two layers of geotextile were attached to the back of the tunnel face piston, both of which were half-circles with a diameter of 45 mm. Test SatP-1B did not have the layers of geotextile. A photograph of the tunnel face is giving in Figure 3.1.

The strongbox used for the tests described in this chapter is the C-CORE “plane-strain” strongbox described in Section 2.4.1 of this dissertation. The only changes made to perform the saturated sand and clay cap tests were the addition of an air-water cylinder. The purpose of this additional apparatus is to apply a trapezoidal pressure distribution onto the inside of the tunnel face. This was accomplished by supplying air pressure in addition to the self-weight pressure of the water in pressure lines leading to the rubber balloon inside the tunnel. This trapezoidal internal face pressure was in all saturated sand and clay cap tests.

3.2.2 Boundary conditions
The strongbox consisted of three aluminium walls, and one clear acrylic wall. Sheets of 6.4 mm thick laminated glass were used to cover the acrylic window and the aluminium wall opposite to the window. Interface friction tests show that this glass and the sand used has an angle of internal
friction of approximately 8°. No glass was used on the sidewalls adjacent to the acrylic window. These boundaries are sufficiently far away from the area of interest, which is the middle third of the strongbox.

A piece of glass was attached to the aluminium tunnel face to reduce friction of sand against it. Again, the measured interface friction angle between the glass and sand was shown to be 8°. The idealised stresses acting on the tunnel face, for a typical saturated sand test are shown in Figure 3.5. Outside of the tunnel, the total soil overburden stress starts at zero at the ground surface and increase linearly with depth, added with the pore water pressure (the ground water table is at the ground surface). Inside of the tunnel, the self-weight water pressure in the balloon is surcharged with air pressure to give a trapezoidal face pressure to resist the applied soil pressure. As the small annulus around the balloon inside of the tunnel is it is hydraulically connected via brass PPT porous stones in the face to the outside pore fluid, this small region contains water pressure at the pressure of the pore fluid.

3.2.3 Model preparation and testing processes
Preparation of the clay layer began by mixing kaolin clay with water to form a slurry with a moisture content of approximately 120%. The clay slurry was then placed inside a model container and one-dimensionally consolidated in a loading frame (Figure 3.2) in increments to a total consolidation pressure of 500 kPa. The equilibrium conditions at each loading increment were observed by monitoring the change in sample volume. After the final load increment had been reached, and consolidation was 90% complete at that increment, the sample was unloaded and allowed to swell back in small increments until a pressure of only 60 kPa remained. The total duration of the process was approximately three weeks. Once unloaded to a stress of 60 kPa, excess water was removed from the top of the sample, and the clay was completely unloaded.
This gave the clay sample a negative pore water pressure of approximately 60 kPa. The clay was then removed from the consolidation container, and was cut into blocks of approximately 870 mm by 290 mm in plan, by 50 mm thick. The clay pieces were then covered in plastic wrap to maintain the moisture content, and set aside until ready to be used.

The model preparation continued by constructing the model tunnel, as explained in Section 2.4.2.1. The pressure tubing from the rubber balloon in the tunnel was then attached to a valve on the strongbox wall. This valve was then directly attached to the air-water cylinder (partly filled with de-aired water), and then to an air pressure regulator-valve system on a portable air supply, shown in Figure 3.3b. The pressure regulator on the portable air supply was then set to 130 kPa, and all valves were slowly opened. Water then filled the rubber balloon thereby locking the tunnel face in place.

The sand was then pluviated into the strongbox in accordance to Section 2.4.2.3 of this dissertation. The density of the sand is governed by the height of the strongbox under the frame. For the tests described in this chapter, the target relative density of the sand was approximately 80% throughout the entire sample. This was obtained by placing the strongbox on a series of supports that raised the bottom to approximately 450 mm off the pluviation room floor. The average density and unit weight for each test is provided in Table 3.2.

The models were then saturated by attaching a saturation cylinder, filled with deionised water, to the drainage ports in the bottom of the strongbox. A layer of geotextile was placed on the model ground surface, followed by an approximately 50 mm thick layer of coarse sand, then several weights. This surcharge overburden pressure served to decrease saturation time, while helping to
prohibit a quick condition of the sand as it was being saturated. Once the sand was fully saturated, the flow of water into the strongbox was stopped, and the soil overburden surcharge was removed from the ground model surface. The process is shown in the photograph in Figure 3.3. At this point, the saturated sand tests were ready for testing.

However, in the case of clay tests, additional steps were taken to create a clay layer on top of the saturated sand. Once saturation was completed and the surcharge was removed from the ground surface, the stored clay was prepared to be placed in the model. Sand that was dyed with a waterproof black ink was gently blown onto the face of the clay block that would be placed against the strongbox window. This would provide enough contrast to use GeoPIV to track movement in the clay layer. The front and back of the strongbox were then lightly greased with petroleum jelly. A damp piece of geotextile was then placed on the clay block, and an aluminium suction plate was placed onto the clay block. This suction plate, shown in Figure 3.4 had a series of grooves attached to a pressure fitting. A vacuum was then drawn, and the clay block was then hoisted and gently placed into the strongbox. The clay layer was placed directly against the viewing window, and the small gaps around the remaining three sides were backfilled with sand so that they would act as side drains while preventing the “slumping” action of the clay in these areas.

Once the model was completely prepared and ready to test in the centrifuge, an aluminium top plate was bolted to the top of the strongbox, in order to reinforce all walls of the strongbox for flight in the centrifuge. Next, an array of conventional displacement transducers (linearly variable differential transformers, LVDTs) was then mounted to the top plate, in order to monitor consolidation of the clay during centrifuge spinup. The strongbox was then carefully transported
to the centrifuge chamber and placed into the centrifuge swing. All data measuring devices were attached to the data acquisition computer, and secured for flight in the centrifuge. The air supply regulator to the balloon was switched from the portable air supply tank to the air supply mounted to the centrifuge swing. This air supply was controlled by an electronic regulator in the centrifuge control room. The centrifuge chamber was then locked, and the testing began by spinning the centrifuge up to the target 50 g acceleration in 10 g steps, and the data recorded. For the saturated sand tests, once the target test acceleration of the centrifuge was attained, the pressure inside the tunnel was decreased at a constant rate until the tunnel face had completely collapsed.

In the case of the clay tests, however, the surface LVDTs were monitored for approximately an hour, allowing for additional consolidation of the clay due to increased self-weight in the centrifuge. Once 90% consolidation was observed, the tunnel pressure was decreased. As soon as the tunnel had completely collapsed, the centrifuge was then spun down, and the strongbox was taken off the centrifuge swing and a post-mortem examination of the model was conducted. The undrained shear strength of the clay layer was then immediately measured using a 19 mm by 28 mm high hand vane, with values given in Table 3.3 along with the moisture content in the clay.

3.3 Results

3.3.1 Saturated sand
To better understand the effect of tunnelling in sand beneath a layer of clay, it is useful to first examine the case of tunnelling in submerged conditions (i.e. saturated sand under the water table) to determine the effect of the pore fluid on the observed tunnel face behaviour. In order to
compare the face balloon pressure-displacement relationship between dry and saturated sand, the results from test Sat-1B is presented in Figure 3.6. In this first test with saturated sand, no special effort was made to prevent the flow of sand particles past the tunnel face into the tunnel once a gap formed. The face displacement behaviour of the model is also presented photographically in Figure 3.7) at three times which correspond to the specific horizontal face displacements denoted on the pressure – displacement curve of Figure 3.6. These results indicate that as the high initial tunnel face balloon pressure (Point O on Figure 3.6) is slowly decreased at a constant rate, very small gradual inward displacement of the tunnel face (less than 0.1 mm) are observed. These incremental displacements were very small and undetectable by the unaided eye, as shown in the photographs in Figure 3.7, until the point of local failure is reached (Point B on Figure 3.6). This point corresponds to the image taken the instant before face stability is lost and the first noticeably large face displacement occurs. That is, before Point B there are smaller movements of the tunnel face with relatively large decreases in internal tunnel face balloon pressure; after Point B there are relatively larger increases in face displacement with smaller deceases in tunnel face balloon pressure.

After local failure occurs (Point B), the face displacements become detectable to the unaided eye, as shown in the photographs in Figure 3.7, where there is a noticeable change in the position of the tunnel face plate between photograph “a” and photograph “c”. Unlike for the tests presented in Chapter 2, however, Figure 3.6 shows only a final horizontal face displacement of approximately 1.4 mm. As shown in the photograph at this face displacement, Figure 3.7, sand can be seen to have travelled into the tunnel and lodged between the tunnel face and the balloon inside the tunnel. Eventually enough sand was deposited behind the face which prohibited any further movement of the face. Any further decrease of the balloon pressure would not result in
any further displacement of the tunnel face, and in fact the balloon lost all contact with the tunnel face (seen in photograph “c” in Figure 3.7).

The reason that sand entered into the tunnel in the saturated case but not the dry case can be explained by the effect of seepage forces being present only in the former. Initially, the pressurised balloon in the tunnel fills the entire volume of the tunnel model, with the exception of a very small void around the sides of the tunnel face. This void is due to the rounded balloon not being able to perfectly fill the square corners of the tunnel model. As the face pressure was decreased in the saturated model, eventually local failure is initiated, and a gap opens between the aluminium face and the void. This enables a rush of water to flow into the tunnel to fill the volume that was once occupied by the void.

The location of the seepage ground loss appears to have occurred a distance away from the tunnel face, in the x-direction as defined in Figure 2.4. In its initial position, the face piston is installed flush with the window so that sand grains could not come between the two during sand placement. An examination of the photographs during the centrifuge testing confirmed that the face piston remained stationary against the window as the centrifuge was spun to the desired gravity factor. Then as the face pressure was decreased, the face moved back slightly but no sand grains were observed to pass between the window and the face (Figure 3.7). The sand that flowed into the tunnel did so along the curved portion of the face. This is due to the larger gaps that formed as the face fell back, due to the geometry of the face being slightly smaller than that of the tunnel (the face had a diameter of 86 mm while the inside diameter of the tunnel is 92 mm). Thus, once the gap between the two became large enough for a sand grain to pass, sand then began to flow into the tunnel.
In Figure 3.8, the 0.1 mm resultant ground movement contour is used to define the extent of ground movement for various known horizontal tunnel face displacements. At relatively small face displacements (roughly 0.02 mm), a local failure mechanism developed, which began at about 10 mm above the tunnel invert and extended about 15 mm ahead of the face and stopped at the tunnel crown. Yet for 0.1 mm of horizontal face displacement, the extent of ground movement began at roughly 10 mm above the tunnel invert, extended about 20 mm ahead of the tunnel face, and extended approximately 5 mm above the crown before closing at the crown.

However, at a higher horizontal face displacement of 1 mm, the extent of ground movement reached the ground surface (Figure 3.8). The extent of ground movement again began at approximately 10 mm above the tunnel invert, extended 25 mm ahead of the tunnel face, reached the ground surface, and then closed directly above the tunnel face at the tunnel. By examining Figure 3.9, a post-test photograph showing the soil surface for test SatP-1B, a ground surface depression was seen, which is highlighted in the photograph for clarification purposes. Again, the final horizontal face displacement was approximately 1.4 mm, which caused this surface ground depression. The depression measured approximately 25 mm in diameter and roughly 15 mm deep, with the centre of the depression located about 28 mm away from the strongbox window. The photograph was taken from the right hand side of the strongbox, so that the strongbox window is orientated on the left side of the photograph. The location of this ground surface depression away from the strongbox window reinforces the postulation that the sand particle flow occurred away from the tunnel (in the x-direction, see Figure 2.4) centreline, as the strongbox window is the assumed axis of symmetry.
However, when measures were employed to intentionally prevent the flow of sand past the tunnel face, the results are markedly different. Test Sat-1 was performed with two layers of damp geotextile attached behind the tunnel face piston. The geotextile served the purpose to prohibit sand particles flowing past the tunnel face into the tunnel, while still maintaining a hydraulic connection between the saturated ground outside the tunnel and the saturated tunnel interior.

For test Sat-1, the internal tunnel face pressure with respect to the average horizontal face displacement at the mid-elevation of the face is plotted in Figure 3.10. A suite of photographs are presented in Figure 3.11. These photographs are of the tunnel model during the centrifuge testing at specific horizontal face displacements on the pressure – displacement curve. Also presented is the progression of the extent of ground movement for test Sat-1 (Figure 3.12), which tracks the progression of the 0.1 mm contour line for various known face displacements along the pressure – displacement curve.

Two points on the pressure displacement curves will be examined, Point B and a horizontal face displacement of 7 mm. As discussed at the onset of this section, Point B on the pressure – displacement curves represents the pressure at the instant before the first tunnel face instability occurs. The horizontal face displacement at this point varies between all tests examined, however it is convenient to discuss this point as a point of failure since ground movements before it were small and localised around the tunnel face, yet after this point ground movements became quite large in comparison, and eventually lead to surface subsidence. Ground movements at a horizontal face displacement of 7 mm are convenient as it is a larger displacement where large ground movements can be comparatively discussed. All ground movements discussed in this chapter are resultant movements.
At low face displacements (Point B = 0.07 mm), there was a localised zone of ground movement ahead of the tunnel face. It started at approximately 10 mm above the tunnel invert, and extended outward about 20 mm ahead of the face, and extended up above the crown by 10 mm. It closed at the crown and did not extend behind the tunnel face. Yet, for a face displacement of 1.0 mm the extent of ground movement began 10 mm above the tunnel invert and extended up and out from there to a distance of 25 mm ahead of the face and a height of 40 mm above the tunnel crown. The bulb-shaped zone closed at the tunnel crown and did not extend behind the tunnel face.

The extent of movement for test Sat-1 at 7 mm of face displacement (Figure 3.12) started about 10 mm above the tunnel invert, and extended ahead of the face by 40 mm until approximately 50 mm below the soil surface, where it extended an additional 15 mm ahead of the face, and only extended about 15 mm behind the face.

As seen in Figure 3.11 (d), which is taken at approximately 7 mm of horizontal face displacement, sand can be seen behind the tunnel face inside the tunnel. However, a review of the test photographs showed that the first movement of sand into the tunnel occurred in between the two consecutive photographs that correspond to a horizontal face displacement of approximately 4 mm and 5 mm on Figure 3.10. For this photograph, only a small amount of sand entered the tunnel model past the face piston, and it did not appear to impede on the subsequent movement of the face piston into the tunnel. With each additional consecutive photograph, more sand was seen to enter the tunnel which was deposited behind the face piston, but again the amount was small and did not appear to have affected the movement of the tunnel face. This flow of sand appeared to have occurred mainly from between the face piston and the strongbox.
window, and possibly to a lesser extent at a distance (in the x-direction, see Figure 2.4) away from the strongbox window.

The addition of the geotextile to the tunnel face appeared to have maintained a hydraulic connection between the saturated ground outside the tunnel with the saturated tunnel interior. It also appeared to have allowed for the same interaction between the tunnel face movement and the ground when compared to the tests examined in Chapter 2, in that it not only allowed for similarly large horizontal face displacements greater than 7 mm, but it the face also experienced the “stick-flip” phenomenon explained in Section 2.5.1. Therefore all remaining tests in saturated ground were conducted with the geotextile adhered to the tunnel face piston.

### 3.3.2 Saturated sand beneath clay

Face balloon pressure – displacement curves are presented for the tests with saturated sand beneath a 50 mm thick layer of over-consolidated clay in Figures 3.13 through 3.16. The shape of the curves is similar throughout the range of C/D examined, where each test exhibited the “stick-flip” phenomenon explained further in Section 2.5.1. The pressures at values of known horizontal face displacement at the mid-elevation of the tunnel face are presented in Table 3.6, where the pressures were measured by a PPT in the tunnel model to an accuracy of 0.1 kPa, and the face displacements were measured by GeoPIV to an accuracy of 0.01 mm. Again, two points on the pressure displacement curves and Table 3.6 will be examined, Point B and a horizontal face displacement of 7 mm. The contours of resultant ground movements are given in Figures 3.17 -3.24.

The pressures measured at Point B varied between 17.3 kPa and 21.6 kPa for the clay cap tests. The horizontal face displacements measured at Point B also varied, ranging from 0.02 mm to 0.21
mm. However, it is noted that the displacements at Point B appear to decrease with an increase in tunnel depth, as shown on Table 3.6. Likewise, the pressures corresponding to the data point closest to 7 mm of horizontal face displacement on Figures 3.13 through 3.16 range from 10.6 kPa to 15.6 kPa.

For face displacements at Point B for the clay cap tests, the extent of the failure mechanism was localised, where the zone of ground movement began roughly 10 mm above the tunnel invert and grew outwards to a distance of 15 mm along the Z-axis and upwards from the invert to a distance at or slightly above the tunnel crown (Figures 3.17 through 3.24). However, for the case of test Cap-0.5, the extent of ground movements is quite different. The zone of ground movement started near the tunnel invert and extended up and out ahead of the face about 25 mm at the elevation of the tunnel crown. At this point lays the bottom of the clay layer where slight yielding of the clay layer occurred, where the extent of ground movement (0.1 mm contour line) extended into the clay layer and just reached the soil surface.

At a horizontal face displacement closest to 7 mm, the ground movement began approximately 10 mm above the invert and extended out, at an angle of roughly 67 to 69 degrees, to 30 mm in front of the face. At approximately the elevation of the tunnel crown, the extent of movement became more vertical, until the clay cap was reached, and also extended slightly behind the tunnel face. It should be noted that the angle at which the extent of ground movement extended out in front of the tunnel face appears to increase with and increase in tunnel depth. Once the ground movements in the saturated sand layer became large enough to influence the overlying clay layer, movements within the clay layer became broader than in sand only, and extended about 100 mm ahead of the tunnel face, and approximately 50 mm behind it. For all clay cap tests, the ground
movement first occurred mainly in the sand layer as the tunnel face displaced. At larger face
displacements, this ground movement created a void at the interface between the clay and sand
layers at larger face displacements (Figures 3.25c through 3.28c) which caused subsequent
movement in the clay, due to the layer yielding as it bridged over the void.

3.4 Discussion

3.4.1 Influence of Pore Fluid
The results of test Dry-1 are given in Chapter 2, however a progression of ground movement plot
for this test is presented here for convenience (Figure 3.30) for comparison with Figure 3.12. At
small horizontal face displacement (Point B), the zones of ground movements were similar for
both tests, as the shape and extent of ground movement were localised around the tunnel face. Of
interest however, is the fact that the tunnel face had a horizontal displacement of 0.07 mm for the
saturated sand test, and 0.26 mm for the dry sand test. As shown in Table 3.4, the initial
calculated horizontal effective stress (at the mid-elevation of the tunnel face) is less for the
saturated sand tests than the dry sand test. Assuming the same coefficients of lateral earth
pressure, the horizontal effective stress for the saturated case is less due to the presence of pore
water pressure. The horizontal stress is directly related to the overburden stress of the soil. When
pore water pressure is present, the effective stress decreases since a portion of that stress is no
longer only carried by just the soil particles of sand, but it is also carried by the pore water.
Therefore since the horizontal effective stress is smaller in the saturated case, the displacement
caused by this smaller stress would be smaller than if the horizontal effective stress was larger (as
in the case of test Dry-1).
At 1 mm of horizontal face displacement, ground movements were slightly different between dry and saturated sand tests. In both tests, the zone of movement starts near the invert, and reached an elevation of 40 mm above the tunnel crown. The zone of movement for the dry sand test was much broader, extending 40 mm ahead of the face, and about 15 mm behind it. The saturated sand test had a much narrower zone of movement that extended only 25 mm ahead of the face with no movement behind the face.

At very large face displacements, the difference between dry sand and saturated sand was greater. At a horizontal face displacement, the extent of movement started at the tunnel invert, and rose directly upwards and outwards away from the face about 75 mm for the dry sand test. It also extended behind the tunnel face; starting at the crown and rising up and out to about 50 mm behind the face. Conversely, the extent of movement for the saturated sand test had more of a bulb shape, as described in Chambon and Corté (1994), which only extended ahead of the face by 40 mm until approximately 50 mm below the soil surface, where it extended an additional 15 mm ahead of the face, and only extended about 15 mm behind the face. As described in Chapter 2 however, the measurement of the ground movements described by Chambon and Corté were not as precise as the dry sand data presented in that chapter; the broader zone of movement is likely due to the funnelling effect described in Chapter 2.

In terms of Mohr-Coulomb yield criteria ($c', \phi'$) Leca and Dormieux (1990) developed upper and lower bound plasticity solutions for a three-dimensional tunnel heading in dry soils. They found that their upper bound solution was in good agreement with the centrifuge tests reported by Chambon and Corté (1989), which are very similar to those presented Chambon and Corté (1994), and were also shown to be in excellent agreement with the dry sand tests described in
Chapter 2. Direct shear tests (ASTM D 3080-90, 1994) performed on the sand used in the present study at the target relative density of approximately 80% yielded a peak friction angle of 47°.

Figure 3.31 shows the upper bound solution with the observed extent of ground movement at failure (Point B) for the tests presented here. The critical geometry for the upper bound solution, given by the dashed lines, coincides reasonably well for the observed extent of soil failure for the dry and saturated sand tests. However, of particular interest is that the failure pressures for the upper bound analytical solution was in accordance with the pressures that Chambon and Corté reported (Leca and Dormieux, 1990). Table 3.5 summarises the failure pressures of Leca and Dormieux and the tests presented here, and fall within the prediction bounds of the analytical upper and lower bound solutions.

For the range of tunnel face displacements around the defined failure pressure (Point B), there appears to be little influence of sand saturation on the shape and extent of ground movements. It can therefore be concluded that the area of importance is just before failure is reached (Point B) which occurs at around 0.1 mm of face displacement. Since the shape and extent of the ground movements around this value of face displacement is largely similar, it appears that saturation of the sand has no effect. However, when the zone of ground movement is sufficiently close to the soil surface, the shape and extent is markedly different, being much thinner in saturated sand. One possible explanation for this difference is that in the saturated sand tests, the effect of this funnelling movement is likely decreased due to the negative excess pore water pressures generated from dilation of the sand when the ground is sheared from tunnel face displacement (i.e. the sand particles just outside of the ground movement chimney). The negative excess pore water pressures would act to hold adjacent sand particles together.
3.4.2 Influence of the clay cap

At local failure of the tunnel face (Point B), the shape and extent of ground movements for both dry sand and clay cap tests is similar. As seen in the contour plots for both tests, it is consistently seen that the ground movements start near the invert, and extend up and out from that point to a distance of 15 mm ahead of the tunnel face, and closing at the tunnel crown. However for test Cap-0.5, the clay layer sits just above the tunnel crown and the local failure in the sand intersects it. It was only in this shallow configuration that the clay cap was close enough to influence the conditions at local failure. For tunnels models deeper than the C/D of 0.5 test (i.e. at a C/D of 1 or greater), the failure mechanism is similar in shape and extent to those for tunnels in just saturated sand. Therefore, it would appear to be reasonable to use the analytical solution provided by Leca and Dormieux (1990) for the clay cap tests presented here. Table 3.5 shows that the clay cap tests are also bound by the upper and lower bound analytical solution.

The relationship between face displacement at the point of local failure (Point B) and tunnel burial depth (expressed as C/D) are plotted in Figure 3.29. Although there is scatter in the data, there exists a general trend of tunnel face displacement at failure (Point B) decreasing as the tunnel depth increases. This is most likely a result of more ground confinement in deeper tunnel (i.e. higher stress levels), which are recognized to increase the stiffness of a soil. In other words, the horizontal earth pressure acting on the face is greater the deeper the tunnel, and if these stresses increase the stiffness of the soil, it is logical that the ground displacement into the tunnel at failure will decrease.

At very large horizontal face displacements of 7 mm, ground movements were similar in shape and extent for the 0.5 C/D ratio in both clay cap and the dry sand tests. However, the void
created at the sand-clay interface causes the clay to “bridge” over the loss of ground directly below it. This is also clearly seen in Figure 3.25c, which is an image taking during the test at which the horizontal face displacement is 7 mm. This is not the case with the tunnel in dry sand, as both the 1 mm and 4 mm contour interval can be approximated by having a V-shape. This shape is due to a loss of ground, experiencing a “funnelling” behaviour of soil into the tunnel.

In deeper tunnels (C/D ratios of 1 or higher), the shape and extent of ground movements at very large face displacements (7mm) are markedly different. As seen in the contour plots for the clay cap tests, the ground movements start near the invert and extent out at an angle to 30 mm in front of the face. At approximately the elevation of the tunnel crown, the extent of movement takes a more direct up-and-down shape, until the clay cap is reached, also extends behind the tunnel face slightly. Once the ground movements are large enough to influence the clay layer, the width of the longitudinal failure trough is exaggerated (i.e. broadened and spread over a much wider area). As shown in Figures 3.20, 3.22, and 3.24, the deformations at the clay surface extend approximately one diameter in front of the tunnel and one-half of a diameter behind it.

The propensity of clay ground to produce wider settlement troughs is well documented in the literature. The clay tests of Mair (1979) illustrated that the observed failure surface started at the tunnel invert, propagating upward and outward becoming significantly wider than the tunnel diameter, as opposed to the much narrower “chimney” seen by Chambon and Corté (1989, 1994). Interestingly, Grant and Taylor (1996) found that when tunnelling in clay under a layer of sand that the subsurface settlement trough (at the clay-sand interface) was much greater than the settlement trough at the ground surface. Thus, the superposition of ground movements from one
layer to another is complex since it requires the prediction of movements from other layers which, in turn, are influenced by the properties and behaviours of adjacent soil layers.

3.5 Summary and Conclusions
Experimental techniques and procedures were developed for simulating tunnel face stability in layered ground. The ground movements associated with decreasing tunnel face pressure were reported for six geotechnical centrifuge experiments. Two of these experiments were performed for a tunnel in saturated sand at a relative density of approximately 80%, and C/D=1.0. The four other experiments were performed at various tunnel depths (C/D = 0.5, 1.0, 1.5, and 2.0), and in saturated sand (approximately 80% relative density), however the top 50 mm of sand was replaced with a 50 mm thick layer of overconsolidated clay. In all tests, the ground water table was located at the ground surface.

The saturated sand experiments (tests SatP-1B and Sat-1), were performed to determine the effect of saturation in sand. Both experiments had a C/D ratio of 1.0, with the top of the groundwater table at the ground surface. In the first of these experiments (SatP-1B) there was no attempt made to prevent the flow of sand particles past the tunnel face. Because of this, a large amount of sand was deposited behind the face as the pressure inside the tunnel was reduced, which prohibited further movement of the face into the tunnel, regardless of any further decrease in face pressure. The flow of sand likely occurred from the inflow of groundwater into the tunnel as the water flowed to fill the volume inside the tunnel that was previously filled by the balloon. The inflow of sand occurred away from the centreline of the tunnel axis (x-axis), and the full effect of ground movements could not be effectively examined because the sand deposited inside the tunnel prohibited further tunnel face displacement regardless of and loss in tunnel face pressure.
However, when effort was made to prevent the flow of sand into the tunnel (test Sat-1), ground movements with tunnel face displacement could be effectively examined. The tunnel face plate moved in a similar fashion as seen in the dry sand tests, exhibiting the same stick-flip phenomenon discussed in Chapter 2. The effect of sand saturation appeared to have no effect for the small tunnel face displacements leading up to local failure (Point B). However at large horizontal face displacements (on the order of 7 mm), the shape and extent of the zone of ground movement was different between Sat-1 and Dry-1, with the zone of ground movement being much thinner in the saturated sand test. The shape of the saturated sand tests resembled the bulb shape seen in Chambon and Corté (1989, 1994). This difference was likely due to the influence of negative excess pore water pressures which could only be generated in the tests with pore fluid.

The ground response of tunnelling in saturated sand beneath a layer of overconsolidated clay was examined in tests Cap-0.5, Cap-1, Cap-1.5 and Cap-2. At small face displacements (Point B) the clay layer appeared to have no influence on the ground movements for sufficiently deep tunnels (C/D≥1.0) when compared to the tunnels in dry sand at the same tunnel depths. Yet for tunnels with half a diameter of cover (C/D=0.5), the clay layer did have an effect on the shape of the ground movements at failure. For such sufficiently shallow tunnels, there was some subsidence in the clay layer above the crown, when compared to the dry sand test of the same tunnel depth, where the zone of ground movement was only localised around the tunnel face. However, for larger horizontal face displacements (7 mm), the sufficiently shallow tunnels (C/D=0.5) exhibit great movement within the sand (> 4 mm), which created a void at the clay-sand interface that the clay layer seemed to bridge over. Likewise for deeper tunnels (C/D≥1.0) the clay cap appeared to influence the magnitude of ground movements at the soil surface compared to the dry sand tests.
In the clay cap tests, the zone of ground movements was similar in shape and size to the dry sand tests until the clay layer was reached, where movements within the clay layer were broad extending 100 mm ahead of the tunnel face. This finding was expected as Mair (1979) experienced similar results for his experiments of tunnels in clay.

Finally, as the zone of soil failing at the instant of tunnel face support (Point B) were so localised, the clay cap was observed to have little to no influence on the face pressure at collapse. In all cases, the pressures at tunnel failure (Point B) were well bounded by the upper and lower bound plasticity solutions developed by Leca and Dormieux (1990), but significantly higher than reported by Chambon and Corté (1989).
Table 3.1: Summarization of centrifuge tests; all tests performed at 50g.

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Test Name</th>
<th>C/D Ratio</th>
<th>Test Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry sand</td>
<td>Dry-2</td>
<td>2.0</td>
<td>Rubber balloon filled with air, tunnel face does not have geotextile or porous elements, talcum powder for used lubrication</td>
</tr>
<tr>
<td></td>
<td>Dry-1.5</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dry-1</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dry-0.5</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Saturated sand</td>
<td>SatP-1B</td>
<td>1.0</td>
<td>Rubber balloon filled with water, tunnel face includes geotextile and porous elements, no talcum powder used for lubrication</td>
</tr>
<tr>
<td></td>
<td>Sat-1</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Clay Cap</td>
<td>Cap-2</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cap-1.5</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cap-1</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cap-0.5</td>
<td>0.5</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2: Measured densities and calculated unit weights for the sand.

<table>
<thead>
<tr>
<th>Test</th>
<th>Ave Bulk Density (kg/m³)</th>
<th>$\gamma_d$ (kN/m³)</th>
<th>$\gamma_{sat}$ (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cap-0.5</td>
<td>1551.8</td>
<td>15.2</td>
<td>19.3</td>
</tr>
<tr>
<td>Cap-1</td>
<td>1521.9</td>
<td>14.9</td>
<td>19.1</td>
</tr>
<tr>
<td>Cap-1.5</td>
<td>1571.2</td>
<td>15.4</td>
<td>19.4</td>
</tr>
<tr>
<td>Cap-2</td>
<td>1503.0</td>
<td>14.8</td>
<td>19.0</td>
</tr>
<tr>
<td>SatP-1B</td>
<td>1534.8</td>
<td>15.1</td>
<td>19.2</td>
</tr>
<tr>
<td>Sat-1</td>
<td>1540.2</td>
<td>15.1</td>
<td>19.3</td>
</tr>
<tr>
<td>Test</td>
<td>C/D</td>
<td>Avg. Cu (kPa) in clay layer</td>
<td>Avg. moisture content (%) in clay layer</td>
</tr>
<tr>
<td>---------</td>
<td>-----</td>
<td>----------------------------</td>
<td>----------------------------------------</td>
</tr>
<tr>
<td>Cap0.5</td>
<td>0.5</td>
<td>7.6</td>
<td>38.5</td>
</tr>
<tr>
<td>Cap-1</td>
<td>1.0</td>
<td>10.9</td>
<td>40.4</td>
</tr>
<tr>
<td>Cap-1.5</td>
<td>1.5</td>
<td>9.1</td>
<td>40.8</td>
</tr>
<tr>
<td>Cap-2</td>
<td>2.0</td>
<td>8.7</td>
<td>42.3</td>
</tr>
</tbody>
</table>

Table 3.3: Average undrained shear strength and moisture content in the clay layer.
<table>
<thead>
<tr>
<th>Test</th>
<th>Pressure at Point B (kPa)</th>
<th>Calc. press. (kPa)</th>
<th>Horiz. face displacement at Point B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( \sigma'_h )</td>
<td>( \sigma'_{ha} )</td>
</tr>
<tr>
<td>Dry-0.5</td>
<td>18.8</td>
<td>22.8</td>
<td>11.8</td>
</tr>
<tr>
<td>Dry-1</td>
<td>13.2</td>
<td>33.5</td>
<td>17.3</td>
</tr>
<tr>
<td>Dry-1.5</td>
<td>14.8</td>
<td>45.9</td>
<td>23.7</td>
</tr>
<tr>
<td>Dry-2</td>
<td>18.1</td>
<td>55.5</td>
<td>28.7</td>
</tr>
<tr>
<td>Cap-0.5</td>
<td>17.3</td>
<td>13.9</td>
<td>7.2</td>
</tr>
<tr>
<td>Cap-1</td>
<td>20.3</td>
<td>20.9</td>
<td>10.8</td>
</tr>
<tr>
<td>Cap-1.5</td>
<td>21.6</td>
<td>28.2</td>
<td>14.6</td>
</tr>
<tr>
<td>Cap-2</td>
<td>18.6</td>
<td>34.8</td>
<td>18</td>
</tr>
<tr>
<td>SatP-1B</td>
<td>15.4</td>
<td>20.9</td>
<td>10.8</td>
</tr>
<tr>
<td>Sat-1</td>
<td>15.2</td>
<td>21.1</td>
<td>10.9</td>
</tr>
</tbody>
</table>

Table 3.4: Summary of calculated horizontal effective pressures for at rest and active conditions at the mid-elevation of the tunnel face, compared with pressures at Point B.
### Table 3.5: Summarisation of failure pressures for friction angle of 47° for three tunnels in dry sand, saturated sand, and saturated sand with clay cap (C/D = 1.0).

<table>
<thead>
<tr>
<th>C/D</th>
<th>Pressure at Point B (kPa)</th>
<th>Predicted Analytical Pressures (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Lower Bound</td>
</tr>
<tr>
<td>Dry-1</td>
<td>13.2</td>
<td>26.8</td>
</tr>
<tr>
<td>Sat-1</td>
<td>15.2</td>
<td>33.8</td>
</tr>
<tr>
<td>Cap-1</td>
<td>20.3</td>
<td>33.4</td>
</tr>
</tbody>
</table>

### Table 3.6: Summary of tunnel face pressures at known values of horizontal face displacement for tests in dry sand and test in saturated sand with 50 mm thick clay cap.

<table>
<thead>
<tr>
<th>Test</th>
<th>Face pressures (kPa) at values of face displacement</th>
<th>Horizontal face displacements at Point B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.10 mm</td>
<td>B</td>
</tr>
<tr>
<td>Dry-0.5</td>
<td>21.3</td>
<td>18.8</td>
</tr>
<tr>
<td>Dry-1</td>
<td>32.6</td>
<td>13.2</td>
</tr>
<tr>
<td>Dry-1.5</td>
<td>23.6</td>
<td>14.8</td>
</tr>
<tr>
<td>Dry-2</td>
<td>18.1</td>
<td>18.1</td>
</tr>
<tr>
<td>Cap-0.5</td>
<td>20.5</td>
<td>17.3</td>
</tr>
<tr>
<td>Cap-1</td>
<td>21.0</td>
<td>20.3</td>
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</tr>
<tr>
<td>Cap-2</td>
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<td>18.6</td>
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Figure 3.1: Tunnel face piston.
Figure 3.2: Consolidation of the clay sample.
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Lights Mounting plate
Saturation cylinder

3.3b. Saturation cylinder Standpipe Air-water cylinder
Portable air supply w/ pressure regulator

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For tests with saturated sand only, the clay layer is replaced with 50 mm of saturated sand.
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Chapter 4

Summary, Conclusions, and Recommendations for Future Research

Experimental techniques and procedures were developed to analyse the progression of ground movements with decreasing tunnel face pressure. For the tests presented in this thesis, a pressure-controlled face boundary was imposed at the tunnel face to accurately measure the soil pressures acting on the tunnel face. The pressure-controlled tunnel face exhibited a stick-flip phenomenon as it was allowed to displace into the tunnel. As a result of this movement, to correlate a zone of ground movement measured using GeoPIV software with a tunnel face displacement, the horizontal component of face displacement was averaged at the mid-elevation of the face since this is the point of the face where the variability in relative (top versus bottom movement) movement was minimal. This displacement was then correlated to face pressure and resultant ground movement.

4.1 Dry sand

Four tests were performed at various cover to depth ratios (C/D of 0.5, 1.0, 1.5 and 2.0) in dry sand with a target relative density of approximately 80%. The dry sand test results were then compared to the physical results of Chambon and Corté (1994) and the analytical models of Leca and Dormieux (1990).

For each dry sand test, the relationship between face balloon pressure and horizontal face displacement averaged around the mid-elevation of the tunnel face was examined. From this relationship, the instant before tunnel face instability (Point B) was determined for each test. In
dry sand, it was determined that there was no influence of tunnel depth on the face balloon
pressure at this point. Also when the resultant ground movement contours were plotted, it was
found that ground movements were small and localised around the tunnel face at this instant. In
fact ground movements before this point were small and localised around the tunnel face, but
ground movements after this point occurred more rapidly and became quite large and eventually
reached the ground surface causing surface subsidence. Therefore this point was defined as the
point of tunnel failure. Since the ground movements at the point of face instability were all
similar in shape and extent, it was deemed that the fashion in which the tunnel face displaced into
the tunnel had no influence on the behaviour of the tunnel at the onset of collapse.

For small face displacements the ground movements were small and localised around the tunnel
face, regardless of tunnel depth. However, for large tunnel face displacements the zone of ground
movements had a bulb shape failure mechanism that reached the ground surface, where sand
within the failure mechanism flowed into the tunnel. Due to the lack of cohesion in the dry sand
tests, there was movement of ground outside of the failure mechanism that moved in towards the
tunnel.

Chambon and Corté (1989, 1991, 1994), who performed similar tests in dry sand, found that
tunnel face pressure at the moment of tunnel face instability to be in the range of 3.5 to 4.0 kPa
for C/D ratios from 0.5 to 2.0. They also presented post-instability ground failures that showed
ground movements to be bulb-shaped and reached the ground surface, creating surface
subsidence. These ground movements were due to the complete collapse of their tunnel face
boundary.
Leca and Dormieux (1990) compared their analysis to results from Chambon and Corté (1989) results and found that their upper bound solutions compared well to the reported centrifuge data of Chambon and Corté, while the lower bound solutions yielded a failure pressure significantly higher. They also predicted a geometry which only coincided well with Chambon and Corté’s horizontal extent of ground movement in front of the face; however it only vertically extended to approximately the tunnel crown, which is not what was found by Chambon and Corté’s results.

The tunnel face pressures at initial instability reported by Chambon and Corté (1989, 1991, 1994) were significantly less than the pressures reported here. The difference is likely due to the initial tunnel face conditions; Chambon and Corté’s tests started at the active earth pressure, which suggests that movement of the tunnel face has occurred during centrifuge spinup. This means that soil arching (soil stress redistribution) could have occurred creating a smaller earth pressure acting on the tunnel face. This was not found to be the case with the tests reported here. Also, the pressures at initial instability, for the tests reported here, were found to be in between the upper and lower bound pressures predicted by Leca and Dormieux (1990).

Ground movements due to tunnel face pressure loss were recorded for six instances for the tests reported here (Points A-F). The ground movements associated with initial instability with the tunnel face were found to be localised around the tunnel face, and coincided well with the zone of ground movement predicted by Leca and Dormieux (1990). However, these movements were not similar to those reported by Chambon and Corté (1994). Rather, Chambon and Corté’s post-instability ground movements correspond well with the points of higher tunnel face displacement (Points E and F), where the zone of ground movement reached the ground surface.
4.2 Unsaturated sand
Likewise, the effect of unsaturated sand conditions was not found to have an effect on the pressures at which initial instability of the tunnel face occurred, but rather it was found to have an influence on the extent of the zone of ground movement. The apparent cohesion and increase in the sand’s strength envelope in the unsaturated sand tests created a thinner zone of ground movement a head of the tunnel face in each of the tests examined when compared to the results of the dry sand tests. In the case of deeper tunnels (C/D=2.0), the zone of ground movement didn’t even reach the ground surface. This is especially important in the field because for the smaller ground movement that occurs, the smaller the volume of ground loss on the surface in the event of tunnel face collapse, which limits potential damage to surface structures and utilities to a smaller zone. For deeper tunnels, this could also provide added safety for surface structures and utilities in such an event. Also, the balloon pressures at initial instability, for the unsaturated sand tests, were found to be bound by the upper and lower bound pressures predicted by Leca and Dormieux (1990).

4.3 Saturated sand
Two experiments were performed to determine the influence of sand saturation at one tunnel depth (C/D=1.0). Both tests were performed at a target relative density of approximately 80%. The saturated sand experiments (tests SatP-1B and Sat-1), were performed to determine the effect of saturation in sand. Both experiments had a C/D ratio of 1.0, with the top of the groundwater table at the ground surface. In the first of these experiments (SatP-1B) there was no attempt made to prevent the flow of sand particles past the tunnel face. Because of this, a large amount of sand was deposited behind the face as the pressure inside the tunnel was reduced, which prohibited further movement of the face into the tunnel, regardless of any further decrease in face pressure. The flow of sand likely occurred from the inflow of groundwater into the tunnel as the water
flowed to fill the volume inside the tunnel that was previously filled by the balloon. The inflow of sand occurred away from the centreline of the tunnel axis (x-axis), and the full effect of ground movements could not be effectively examined because the sand deposited inside the tunnel prohibited further tunnel face displacement regardless of and loss in tunnel face balloon pressure.

However, when effort was made to prevent the flow of sand into the tunnel (test Sat-1), ground movements with tunnel face displacement could be effectively examined. The tunnel face plate moved in a similar fashion as seen in the dry sand tests, exhibiting the same stick-flip phenomenon discussed in Chapter 2. The effect of sand saturation appeared to have no effect for the small tunnel face displacements leading up to local failure (Point B). However at large horizontal face displacements (on the order of 7 mm), the shape and extent of the zone of ground movement was different between Sat-1 and Dry-1, with the zone of ground movement being much thinner in the saturated sand test. The shape of the saturated sand tests resembled the bulb shape seen in Chambon and Corté (1989, 1994). This difference was likely due to the influence of negative excess pore water pressures which could only be generated in the tests with pore fluid. Therefore there was no additional funnelling of sand from outside the ground failure mechanism in towards the tunnel for a saturated dense sand.

4.4 Saturated sand beneath clay

Experimental techniques and procedures were developed for simulating tunnel face stability in layered ground. The ground movements associated with decreasing tunnel face pressure were reported for six geotechnical centrifuge experiments. Four experiments were performed at various tunnel depths (C/D = 0.5, 1.0, 1.5, and 2.0), in saturated sand (approximately 80% relative density), however the top 50 mm of sand was replaced with a 50 mm thick layer of overconsolidated clay. In all tests, the ground water table was located at the ground surface.
The ground response of tunnelling in saturated sand beneath a layer of overconsolidated clay was examined in tests Cap-0.5, Cap-1, Cap-1.5 and Cap-2. At small face displacements (Point B) the clay layer appeared to have no influence on the ground movements for sufficiently deep tunnels (C/D ≥ 1.0) when compared to the tunnels in dry sand at the same tunnel depths. Yet for tunnels with half a diameter of cover (C/D = 0.5), the clay layer did have an effect on the shape of the ground movements at failure. For such sufficiently shallow tunnels, there was some subsidence in the clay layer above the crown, when compared to the dry sand test of the same tunnel depth, where the zone of ground movement was only localised around the tunnel face. However, for larger horizontal face displacements (7 mm), the sufficiently shallow tunnels (C/D = 0.5) exhibit great movement within the sand (> 4 mm), which created a void at the clay-sand interface that the clay layer seemed to bridge over. Likewise for deeper tunnels (C/D ≥ 1.0) the clay cap appeared to influence the magnitude of ground movements at the soil surface compared to the dry sand tests. In the clay cap tests, the zone of ground movements was similar in shape and size to the dry sand tests until the clay layer was reached, where movements within the clay layer were broad extending 100 mm ahead of the tunnel face. This finding was expected as Mair (1979) experienced similar results for his experiments of tunnels in clay.

Finally, as the zone of soil failing at the instant of tunnel face support (Point B) were so localised, the clay cap was observed to have little to no influence on the face pressure at collapse. In all cases, the pressures at tunnel failure (Point B) were well bounded by the upper and lower bound plasticity solutions developed by Leca and Dormieux (1990), but significantly higher than reported by Chambon and Corté (1989).
4.5 Recommendations for future research

The results reported in this chapter are applicable only for the specific conditions (i.e. model materials, tunnel depth, sand backfill, geometry, and boundary conditions) examined in the experiments. The results presented here may be used by other researchers to calibrate numerical analyses which can help to better quantify the progression of ground displacements with loss of tunnel face stability in a range of field conditions. Further studies continuing from this research could look at different mixed face soft ground tunnelling conditions, such as soil stratum interface at an elevation within the tunnel face horizon. Other studies could examine the influence of increasing face pressures after tunnelling failure occurs to examine the shape and extent of ground movements in a range of ground conditions. This data would be especially important to the tunnelling community to help examine ground movements after a problem with a TBM has encountered, and then corrected.
References


