VITRIFIED CLAY PIPE JOINT BEHAVIOUR UNDER DIFFERENTIAL GROUND MOVEMENT

By

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A thesis submitted to the Department of Civil Engineering

In conformity with the requirements for the degree of Master of Applied Science

Queen’s University

Kingston, Ontario, Canada

(September 2015)

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Abstract

Field-scale experiments were used to investigate the behaviour of pipe joints connecting four segments of vitrified clay pipe subjected to differential ground movement. Firstly, linear potentiometers, inclinometers and strain gauges were installed to monitor joint rotations, joint axial compression or extension, and pipe strain. An articulation test using a servo-controlled hydraulic actuator was then employed to assess the instrumentation functionality and accuracy. The second stage of the project involved design and construction of a field-scale test chamber to simulate a normal ground fault. This facility employs manually operated screw jacks to displace a moveable floor. A test using olivine sand was conducted with floor displaced downwards by 122 mm, and this demonstrated that the new test chamber simulates normal ground faulting safely and within the design requirements. Thirdly, two tests were conducted where the pipe assembly was buried at different depths in the olivine sand and subjected to differential ground motion resulting from 30 mm of vertical floor displacement. Analysis of the experimental results included examinations of how joint rotations, joint axial compression or extension, and pipe strains increased with floor (i.e. fault) displacement, how burial depth influenced the observed behaviour, and how measured joint responses compared to two simple geometrical models.

The relationship between joint rotation and vertical floor (i.e. fault) displacement was almost linear, and close to a simple trigonometric relationship developed in an earlier project. Joint axial compressions/extensions were non-linear functions of vertical floor (fault) displacement. This included a transitional event, where after the joint experienced a small amount of axial compression or extension that axial response reversed direction and much larger (i.e. dominant) extension or compression developed. Pipe burial depth had little effect on joint rotations, but a substantial effect on joint compressions or extensions. While total net axial extension was similar
to that predicted using a simple geometric relationship, the values at individual joints were substantially higher. Maximum bending strains of 114 and 300 microstrain were measured along the pipe segments at 0.6m and 1.2m burial depth, respectively. The larger value is about 1/3 of the tensile strain capacity of vitrified clay.
Acknowledgements

This research project was conducted under the supervision of Dr. Ian D. Moore. I am forever grateful for the time, effort and enthusiasm that he had contributed to this project. In addition, I am thankful for Dr. Moore patience and guidance during my two year research experience at Queen’s University.

The research project was conducted in the GeoEngineering buried infrastructure laboratory at Queen’s University, which is funded by the Canadian Foundation for Innovation (CFI) and Natural Sciences and Engineering Research Council of Canada (NSERC). Any opinions, findings, conclusions or recommendations expressed in this thesis are those of the author and his collaborators and do not necessarily reflect the views of the sponsors.

I appreciate Logan Clay Pipe for the donation of all six clay pipe specimens.

I would like to extend my personal thanks to individuals for their assistance during my research: Mr. Graeme Boyd, Mr. Brian Westervelt, Ms. Jane Peter, Mr. Pengpeng Ni, and Dr. David Bercerril Garcia. This project would not have become a reality without them.

Lastly, I am grateful for my parents’ emotional and moral support during my studies. A very special personal thank you to my partner, Joanna Chou, for her encouragement and patience throughout this academic journey.
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Chapter 1 Introduction

1.1. Description of problems

Underground infrastructure like pipelines provides crucial lifelines for modern society. These lifelines provide water, sewage control, natural gas, and electricity to billions. Natural disasters or human activities can threaten these lifelines. Differential ground movement is one of these disruptors to underground pipelines. Differential ground movement can be caused naturally (e.g. due to earthquake faulting) but it can also be caused by human activities. These human activities can include ground subsidence resulting from excavation, mining, and groundwater withdrawal. Many sewers have been constructed using vitrified clay pipes, since vitrified clay is chemically inert and is immune to corrosion. The national averages of vitrified clay pipe utilization for storm and sanitary collection for both Canada and the United States of America are 16% and 56%, respectively, Allouche and Freure, (2002). When vitrified clay pipelines are subject to differential ground movements, two different outcomes may be possible:

i. rotation of the joints and/or the development of shear forces across the joints could result in failure of the gaskets within the joints to prevent leakage of groundwater into the sewer; that water inflow increases the volume of wastewater requiring treatment; water inflow can also cause erosion of soil from around the pipe; the resulting reduction of soil support around the pipe segment causes redistribution of earth loads on the pipe, and the increases in circumferential bending moment can induce longitudinal fractures which further increase the backfill erosion and the amount of wastewater requiring treatment (see Moore, 2008, for detailed discussion)
ii. Longitudinal bending moments can also develop along the pipe segments, and these can lead to ring fractures; ring fractures also increase ingress of groundwater, backfill erosion, and the amount of wastewater requiring treatment.

Research on the longitudinal soil-pipe interaction that occurs when vitrified clay sewage pipelines are subject to differential ground movements can therefore assist in controlling the amount of water requiring treatment and improve our understanding of the deterioration processes that control the long-term performance of sewer pipelines. The response of a vitrified clay sewer pipeline to differential ground movement is therefore the focus of attention in this thesis.

1.2 Project Objectives

The main objective of this research project is to monitor joint behaviour of a pipe assembly under differential ground movements associated with a normal (i.e., 90° to the horizontal plane) ground fault (many different ground deformation patterns are possible, but the normal ground fault is chosen since it involves a well-defined ground-motion that can be reproduced in the laboratory). The pipe assembly consists of four pipe segments connected by three pipe joints. To achieve the main objective of this project, several steps are required:

- Development of a joint monitoring system with various instruments including Linear Potentiometers and Inclinometers. This system is designed to monitor joint rotations and axial (horizontal) joint movements during the differential ground movement event.
- Application of strain gauges on the external surface of the entire pipe assembly. These gauges measure longitudinal strains along and around the pipe segments as the assembly is subjected to differential ground movement.
• An articulation test of this monitoring system to verify that the instrumentation can accurately record displacement and rotation data, and deemed suitable to monitor joint behaviours for subsequent buried field-scale tests.
• Design and construction of a test chamber to simulate the effect of a normal ground fault on a block of soil and any buried pipeline present within; the test chamber has to permit imposition of vertical differential ground movement of sufficient magnitude to induce significant rotations at joints and longitudinal bending moments in the pipe segments (for the work of this thesis, at least 25 mm of vertical displacement is required). The test chamber must also be able to contain the soil mass inside the test chamber without ‘leakage’ during the test preparation and imposition of the ground displacement.
• Conduct of field-scale tests on the pipe assembly buried at two different depths, 1.2 m (designated as the deep burial case) and 0.6 m (designated as the shallow burial case).
• Analysis of data from both burial tests to quantify the relationships between joint rotations and axial translations and the amount of vertical ground movements.

1.3 Literature Review

Only a few studies have been performed which discuss the effects of differential ground movement on pipe joints. Rajani and Tesfamariam (2004) used beam-on-elastic-spring analysis to perform a sensitivity study examining how thermal effects and partial ground support resulting from erosion influence axial, flexural, and circumferential stresses in jointed cast iron, ductile iron and polyvinyl chloride (PVC) water pipes. Balkaya et al. (2012) used three dimensional finite element analysis to examine the impact of erosion voids under jointed PVC pipelines. However, neither of those computational studies established the performance of their analysis
procedures in relation to test data, and their studies focused on water pressure pipelines not the gravity flow clay sewer pipes examined here.

Becerril García and Moore (2013, 2014, 2015) examined vehicle load effects on jointed thermoplastic, corrugated steel and reinforced concrete culvert pipelines, respectively, and Wang and Moore (2014, 2015) developed computer models for the shear forces and rotations that develop across gasketed bell and spigot joints as a result of vehicle loads and changes in soil stiffness. However, those studies focussed on culverts having inner diameters of from 0.6 m to 1.2 m, and they did not consider the impact of differential ground movements.

Kim et al. (2012) conducted an experiment using the Large-Scale Lifelines Testing Facility at Cornell University to examine the effect of a strike-slip fault on a reduced-scale model of a jointed concrete pipeline, where the fault traversed at an angle 65° to longitudinal axis of the pipes. In addition to examining a different kind of differential ground movement, a model test could only be expected to provide unreliable evidence for use in assessing the potential for joint leakage (since gasketed joint behaviour is not well understood and it is currently difficult or impossible to design model joints that reflect the leakage behaviour of production pipes). Instead, the objective of the Cornell experiment was identification of failure modes associated with very large ground movements (those associated with a large ground rupture during an earthquake) rather than the joint movements that might induce leakage resulting from smaller movements (the focus of the current thesis). Their work, however, provides valuable guidance on the use of linear potentiometers mounted inside the pipe joint to monitor the joint movements.

Buco et al. (2008) studied the mechanical behaviour of joints in reinforced concrete sewer pipes. Two pipe segments were joined, vertically supported and horizontally restrained while hydraulic
jacks applied loading to the pipe system. The pipe joint was subjected to axial compression, bending and vertical shearing to study its mechanical behaviour. They then developed simplified rheological models to characterise the axial compression and transverse shear response of the joint. However, they did not test buried jointed pipelines, and did not consider vitrified sewer pipes, the impact of differential ground movements, or more than a single joint connecting two pipe segments.

The goal of the experiments presented in this thesis is to provide details of the response of production pipes with gasketed bell and spigot joints to differential ground movements. Those experiments can:

a. show how both axial movement and joint rotation develop in a full-scale gasketed pipeline;

b. illustrate how longitudinal bending moments develop along the pipe barrels;

c. provide information for use in evaluating and developing three dimensional computer models of jointed pipeline behaviour

d. be a starting point for subsequent experimental studies where joint leakage is monitored and other pipelines are investigated under differential ground movements.

1.4 Thesis Format

This thesis is presented in the manuscript format as outlined by the School of Graduate Studies at Queen’s University. The abstract is followed by the introduction and background in Chapter 1 of this Thesis. The three following chapters: Chapters 2, 3, and 4 are the manuscripts. The findings of this thesis are summarized in Chapter 5. Chapter 2 focuses on the joint instrumentation scheme to monitor joint behaviour and longitudinal strain on the pipe barrels. An in-air
articulation test and its results are also presented in Chapter 2. Chapter 3 discusses the development of the test chamber to simulate normal ground faulting (both its design and its construction). A trial experiment conducted in this test chamber is also included in Chapter 3, monitoring the response of the soil when subjected to differential ground movement. In Chapter 4, two buried pipeline tests are presented and discussed. Joint behaviours from both tests are presented and analyzed. Ultimately, Chapter 5 summarizes the research findings and conclusions from all three previous chapters.

1.5 References


Chapter 2 Pipe Samples, Joint Monitoring System, and Full-Scale Articulation Test

2.1. Introduction

This chapter introduces the pipe test samples, the instrumentation scheme developed to monitor joint movements, and the design and conduct of the ‘articulation test’. That test was performed on the assembled, instrumented test pipes at the GeoEngineering Laboratory located at Queen’s West Campus. It involved four vitrified clay (‘rigid’) test pipes, each with an inner spigot diameter of 150 mm, which forms a pipe assembly 6.1 m long. The pipes had never been used previously or exhumed from the ground prior to this research project. The articulation test was carried out to ensure all instruments (Linear Potentiometers and Inclinometers to monitor joint movements, and Strain Gauges to monitor longitudinal bending along each pipe segment) are in working order and able to produce accurate and precise readings, and to confirm the success of calculation procedures used to determine joint rotation and joint extension. The results of that test are therefore presented, and conclusions are drawn regarding the effectiveness of the monitoring system.

2.2. Problem of interest

The problem of interest in this research project is the joint responses and the longitudinal bending that develops along vitrified clay pipes when they are subjected to different ground movement. Differential ground movement (DGM) presents a challenge to pipe infrastructure long term performance and stability. It can be arise from sources such as earthquake faulting, subsidence due to excessive groundwater extraction, settlement from underground excavation, the effect of moisture changes in reactive clays, and due to ground freezing.
Vitrified clay pipe was historically used in sewer construction for over a century, due to it being chemically inert; it is ideal for wastewater drainage. However, over time vitrified clay pipes can fail. One contributing factor to vitrified clay pipe failure is leakage of pipe joints, which can be induced by differential ground movement causing joint rotation beyond the pipe joint’s design limit. Based on ASTM C425, a joint for a 6” vitrified clay pipe has a maximum rotation of 2.4° (ASTM C700-13, 2013). Leakage of the pipe causes serviceability problems, where transmission of fluid becomes less efficient and effective. Leakage of pipe joints can then cause erosion of backfill in the vicinity of the structure, which can cause loss of side-support and increases in circumferential bending moments to the point where longitudinal fractures occur, Moore (2008), typically along the crown, the invert and springlines. Differential ground movements along the pipeline also induce longitudinal bending moments within the pipe segments, and these can lead to ring fractures in the pipe segments. The fracturing of the pipes represents an ultimate limit state.

The current research project features experimental work on an assembly of four vitrified clay pipes, to see the impact of differential ground movements associated with a normal (90° to horizontal plane) ground fault. The current research project examines jointed pipe responses that stay within the limits of serviceability for the pipe joints and which do not result in any ultimate limit state associated with ring fractures resulting from longitudinal bending, or crushing or other damage at the pipe joint.

Following the development of the assembled, instrumented pipe sample in the current chapter, Chapter 3 describes the development of a new test chamber to impose differential ground movements associated with a normal ground fault, and Chapter 4 describes the response of the
instrumented test pipe after it is buried in the test chamber and subjected to those differential ground movements.

2.3. Pipe Sample and Assembly

The vitrified clay pipes used for this research project were manufactured by the Logan Clay Products Company as sanitary sewer pipes. Table 2-1 summarizes the dimensions of each pipe.

Table 2-1 Clay pipe specimen dimensions

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (m)</td>
<td>1.52</td>
</tr>
<tr>
<td>I.D (Spigot/Bell) (mm)</td>
<td>150.0</td>
</tr>
<tr>
<td>O.D (Spigot) (mm)</td>
<td>191.0</td>
</tr>
<tr>
<td>O.D (Bell) (mm)</td>
<td>274.6</td>
</tr>
</tbody>
</table>

The joints of the pipes were O-ring compression joints. Each joint contained a rubber gasket with a cross-sectional thickness of approximately 1.11 cm. Both spigot and bell portions of each joint were coated with polyester. Figure 2-1 provided by Logan Clay Products depicts the joint connection along with the gasket location.

Figure 2-1 Gasket and joint connection (Logan Clay products company, 2014)

A rubber gasket was placed in each joint to accommodate joint movement. In Figure 2-2 displays the rubber gasket as provided by Logan Clay Products.
A total of four clay pipes were used for this project. The pipes were labelled TP1 to TP4 (where TP denotes “Test Pipe”). Figure 2-3 displays the longitudinal layout and labelling of the pipe assembly. There are a total of three joints in this pipe assembly, and the entire pipeline is approximately 6.1 m long.

Figure 2-3 Longitudinal profile of pipe assembly
2.4. **Pipe and Joint Instrumentation**

The instrumentation scheme of this pipe assembly involves Linear Potentiometers, Inclinometers, and Strain Gauges. Linear Potentiometers were used to monitor axial extension/compression of the joints, lateral movement of joints during differential ground movement, and joint rotation relative to the horizontal (due to closing or opening of the joints); The Linear Potentiometer placement scheme was modelled after the approaches employed by Kim et al. (2012) and Buco et al. (2008). The inclinometers placed inside the bell portion of each pipe segment were to monitor inclination in real-time during the experiments, and they also serve as verification of the joint rotations obtained from the LPs. Inclusion of inclinometers was partially inspired by the experimental work of Sim et al. (2012), where accelerometers were attached to the test pipes. Strain gauges adhered to the crown and to the invert on the outside of each pipe segment were used to measure longitudinal strain which could subsequently be used to infer bending moments.

The Linear Potentiometers (LPs) were manufactured by Penny & Giles – the SLS 130 series which measured up to 25 mm mechanical stroke. The repeatability of the instrument reported by the manufacturer was to be less than 0.01 mm. Three LPs placed in a triangular configuration were used to monitor each joint (i.e. nine LPs were used to monitor all three joints). This triangular configuration permitted the assessment of three joint responses:

- Axial joint extension or compression, and
- Joint rotation about the vertical axis, and
- Joint rotation about the horizontal axis
An aluminum rail system from Thorlab was used to mount the linear potentiometers and the final assembly is shown in Figure 2-4.

One LP was placed under the crown within the spigot portion of the pipe, and two other LPs were placed at the haunch positions. The LPs are placed in a triangular configuration to measure displacement of the bell portion of the adjacent pipe segment. The dimensions of the LP layout are provided subsequently in Figure 2-5.

Joint rotation is calculated by trigonometry using haunch LP readings relative to the crown LP. Equation 2-1 shows the calculation of joint rotation as denoted $\theta_{Joint}$ from LP readings.

Measurements are in millimetres. Crown LP is labelled as LP1 and Haunch LPs are labelled as LP2 and LP3. $H$, is the height of the haunch LPs relative to the crown LP is approximately 75.4 mm.

$$\theta_{Joint} = \tan^{-1}\left(\frac{LP_3 - LP_2}{h}\right)$$  \hspace{1cm} (Equation 2-1)
This experiment aims to achieve approximately 1 degree of joint rotation at the middle joint of the pipe assembly (i.e. Joint #2). Based on geometry and length of test pipe, L, (approximately 1.52 m), the degree of joint rotation can be calculated using the height that the joint is lifted based on trigonometry; Equation 2-2 provides the calculation of vertical displacement required to induce an angle of rotation of $\theta_{\text{required}}$:

$$h_{\text{Displacement}} = (L)\tan \theta_{\text{required}}$$  \hspace{1cm} (Equation 2-2)

The vertical displacement required is in millimetres. For this articulation test, the total displacement was set to be 25.4 mm (1 inch), for ease of communication to technical staff. This vertical displacement translated to 0.957 degrees of rotation (i.e. approximately 1 degree).

Figure 2-5 Linear potentiometer layout normal to pipe axis in spigot portion (dimensions are in millimetres)

Figure 2-6 depicts the cross-section schematic of the LP system. A wooden ring was placed in the bell portion of the joint as the contact plate for LPs mounted on the spigot portion. The LPs’
rods protruded outward approximate 20 mm as measured from edge of spigot to connect to the contact plate, which allowed for displacement measurement of the bell.

![Figure 2-6 LP System cross-section schematic (dimensions are in millimetres)](image)

A small scale trial test was conducted in October 2014 for one set of LPs. The trial test confirmed the accuracy of the test configuration. For more details concerning this particular trial test, refer to Appendix B of this thesis.

For the measurement of inclination, four inclinometers were used. The inclinometers were manufactured by Rieker Incorporated; they were liquid capacitive electronic analog sensors. The inclinometers were from the N2 series with a measurement range of +/- 10 degrees with resolution of less than <0.002 degree. An inclinometer was placed inside the bell portion (approximately 5 cm inward) of each joint. Inclinometers were attached to the invert position with Velcro. The inclinometers would measure the inclination of each pipe segment, and given the rigidity of clay pipes, the inclination of adjacent pipe segments could be used to infer the rotation of the joint in the vertical plane.
Longitudinal bending development along rigid vitrified clay pipes which could lead to ring fractures if the extreme fibre strain reached the tensile strain capacity of the material. To monitor local strains, a total of six strain gauges were attached to the outer surface of each test pipe. The strain gauges were manufactured by Kyowa and had 4 mm gauge length with a gauge resistance of 120.0 (+/− 0.4) $\Omega$. Strain gauges are the KFG120 uniaxial series from Kyowa and they are designed for general strain measurements with temperature compensation for steel. Three strain gauges were installed along the crown and oriented longitudinally and another three along the invert; all at even spacing between one another. Figure 2-7 displays locations of all six strain gauges used on each pipe segment measured relative to the bell (dimensions in millimetres).

![Strain gauge locations diagram](image)

Figure 2-7 Strain gauge locations (dimensions are in millimetres)

The installation procedure for the strain gauges is outlined in Appendix A. Adhesive system AE-10 from Vishay Precision Group was used to bond strain gauges to the test pipes, following surface preparation as outlined in instruction bulletin B129-8 (2010) from Vishay Precision Group.
2.5. Test Setup

For the preparation of the test pipe assembly, 27-XL (or “duck butter”) manufactured by Phoenix was used to lubricate the gasket which was then fitted onto the spigot portion of the pipe. Each pair of pipes was then secured together using ratchet straps, with tension applied to force the two pipes together. Figure 2-8 displays the joint after lubrication.

![Figure 2-8 Joint after lubrication](image)

A total station was employed for external displacement measurement of joints; the station was controlled remotely to avoid interference during testing. Total prisms were mounted on top of
test pipes’ joints. Figure 2-9 displays a total station prism on Joint #1 (between Test Pipe #1 and Test Pipe #2).  

![Figure 2-9 Total station prism](image)

The total station measured displacement of joints in all three Cartesian directions (i.e: X, Y, Z). A benchmark reading was taken by the total station before commencement of the articulation test. A reading was taken at the end of every lift step; a lift step was 5 mm of vertical displacement. Readings were recorded remotely to ensure no interference of test setup during testing.

The pipe assembly was oriented in the East-West direction; the total length of pipe assembly was approximately 6.1 metre. It was placed on a gravel surface of relative uniform grade, and it was supported by wooden wedges. Joint #2 was the only lifting point of the pipe assembly; it was lifted by a hydraulic actuator. Joint #1 was at the East end; Joint #3 was at the West end. The test pipe assembly along with actuator is displayed in the Figure 2-10.
Two tests were conducted where the pipe assembly was lifted up at Joint #2; the lifting rate was 2.5 mm per minute. Maximum displacement was set to 25 mm (approximately 1 inch) which was based on simple geometry to provide approximately 1° of rotation. Data from all instruments except total station were recorded by the Data Acquisition System 5000 with the 51000B scanners manufactured by Vishay Measurement Group. The uplifting portion of the test was conducted in 5 steps (for a total of 25 mm); lowering of the pipe assembly back to its original position was completed in 2 steps. Rate of uplift was set to 2.5 mm per minute; the pipe assembly was held steady for approximately 2 to 3 minutes for stabilization and so that total station readings for all three joints were taken. Each lift step (including a total station reading) was approximately 5 minutes. Rate of lowering was also set at 2.5 mm per minute but each lowering step was set at a total of 10 mm of displacement, after which the assembly was held steady for approximately 2 to 3 minutes for stabilization, and total readings for all three joints were obtained. Joint #2 was secured to the hydraulic actuator using a ratchet strap, which was
wrapped on the underside of the bell portion of the joint; this was done to produce an even
distribution of force during lifting/lowering. Figure 2-11 depicts Joint #2 being secured to the
hydraulic actuator.

Figure 2-11 Joint #2 secured to actuator

2.6. Test Results and Discussion

The following sections present results from the articulation tests. Results from inclinometers,
LPs, strain gauges, and total stations will be shown and discussed. A grand total of four
inclinometers, nine LPs and twenty-four strain gauges were used to instrument the four-pipe assembly.

### 2.6.1. Inclinometer Results from Articulation Test

Figures 2-12 and 2-13 are the raw inclination data from the two articulation tests.

![Inclinometer readings during articulation test #1](image)

Figure 2-12 Inclinometer readings during articulation test #1
Table 2-2 denotes inclinometers responsible for monitoring each joint.

<table>
<thead>
<tr>
<th>Joint #</th>
<th>Test Pipe</th>
<th>Inclinometers</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TP1+TP2</td>
<td>I1 and I2</td>
</tr>
<tr>
<td>2</td>
<td>TP2+TP3</td>
<td>I2 and I3</td>
</tr>
<tr>
<td>3</td>
<td>TP3+TP4</td>
<td>I3 and I4</td>
</tr>
</tbody>
</table>

From Figures 2-12 and 2-13, it appears all inclinometers were functional over the course of the tests. All inclinometers were calibrated before conducting the articulation test by using a wooden wheel as shown in Figure 2-14.
An inclinometer is attached to the plate and then calibrated using the data acquisition system. For defining rotation relative to the vertical plane; positive sign denotes clockwise rotation and negative sign vice versa. Figure 2-15 depicts the sign convention for all four inclinometers.
Table 2-3 to Table 2-5 summarize inclinometer readings from the two articulation tests. Values in tables are averaged values in degrees from all four inclinometers. The maximum displacement is 25 mm of vertical displacement during lifting of the hydraulic actuator.

Table 2-3 Inclinometer readings summary for articulation test #1

<table>
<thead>
<tr>
<th>Inclinometers</th>
<th>Base Reading (Degree)</th>
<th>Reading at Maximum Displacement (Degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I1</td>
<td>0.299</td>
<td>-0.044</td>
</tr>
<tr>
<td>I2</td>
<td>-0.058</td>
<td>0.591</td>
</tr>
<tr>
<td>I3</td>
<td>0.174</td>
<td>-0.569</td>
</tr>
<tr>
<td>I4</td>
<td>-0.121</td>
<td>-0.042</td>
</tr>
</tbody>
</table>

Table 2-4 Inclinometer readings summary for articulation test #2

<table>
<thead>
<tr>
<th>Inclinometers</th>
<th>Base Reading (Degree)</th>
<th>Reading at Maximum Displacement (Degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I1</td>
<td>0.200</td>
<td>-0.028</td>
</tr>
<tr>
<td>I2</td>
<td>-0.130</td>
<td>0.544</td>
</tr>
<tr>
<td>I3</td>
<td>0.100</td>
<td>-0.538</td>
</tr>
<tr>
<td>I4</td>
<td>-0.056</td>
<td>-0.013</td>
</tr>
</tbody>
</table>

The difference of the inclinometer readings above yield the rotations experienced by each joint at maximum vertical displacement.
Table 2-5 Total joint Rotation at maximum displacement (inclinometer readings)

<table>
<thead>
<tr>
<th>Joint #</th>
<th>Rotation from Articulation Test #1 (Degree)</th>
<th>Rotation from Articulation Test #2 (Degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.635</td>
<td>0.625</td>
</tr>
<tr>
<td>2</td>
<td>-1.160</td>
<td>-1.126</td>
</tr>
<tr>
<td>3</td>
<td>0.527</td>
<td>0.585</td>
</tr>
</tbody>
</table>

The measured joint rotation from inclinometer readings, particularly at Joint #2, matched expected joint rotation of approximately 1 degree (calculated from geometry) at maximum vertical displacement. In terms of percentage difference, Test #1 yielded a measurement difference of 16% and 12% compared to Test #2. This phenomenon was perhaps due to data noise from the inclinometer readings. From these inclinometer readings which corresponded to the expected joint rotation, it appeared that the inclinometers performed as intended during the two articulation tests.

2.6.2. Linear Potentiometer Results from Articulation Test

Three sets of linear potentiometers (LPs) were used in this full-scale articulation test. The layout and geometry were discussed earlier in this chapter. The purpose of this articulation test was to confirm that LPs were all functioning and providing accurate readings. Figure 2-16 to Figure 2-18 are LP displacement readings from the second set of tests:
Figure 2-16 LP displacement readings joint #1

Figure 2-17 LP displacement readings joint #2
All of the LPs performed in synchronization with actuator stroke displacement, and it appeared that all LPs behaved properly due to their instantaneous responses. However, the crown LP in Joint #2 (J2_LP1) produced high displacements initially beyond the allowable maximum LP translation of 25 mm; it appears that there was a calibration issue with this LP.

The crown LP in Joint #2 was re-calibrated by averaging the calibration records of the other two haunch LPs (i.e. J2_LP2 and J2_LP3). In Figure 2-19 to Figure 2-21 display the comparisons of joint rotations as measured by both LPs and inclinometer during articulation test #2.
Figure 2-19 Joint rotation measurements comparison in joint #1

Figure 2-20 Joint rotation measurement comparison in joint #2
From the comparison figures, it is concluded that both LP and inclinometer measurements produce rotations that are largely in agreement (within 15%) and they can be relied upon for joint rotation measurements in subsequent experiments.

Table 2-6 to Table 2-8 outlined rotation measurements from LPs and inclinometers from Articulation Test #2 at maximum displacement. $\delta_{I-LP}$ denoted the difference of degree of rotation as measured by LPs and inclinometers.

Table 2-6 Rotation measurement difference for LPs and inclinometers at maximum displacement for joint #1

<table>
<thead>
<tr>
<th>Relative to J1_LP2 (Degree)</th>
<th>Relative to J1_LP3 (Degree)</th>
<th>LPs Average (Degree)</th>
<th>Inclinometer (Degree)</th>
<th>$\delta_{I-LP2}$ (Degree)</th>
<th>$\delta_{I-LP2}$ (%) Difference</th>
<th>$\delta_{I-LP3}$ (Degree)</th>
<th>$\delta_{I-LP3}$ (%) Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.635</td>
<td>0.613</td>
<td>0.624</td>
<td>0.625</td>
<td>0.01</td>
<td>1.60</td>
<td>0.012</td>
<td>1.92</td>
</tr>
</tbody>
</table>
Table 2-7 Rotation measurement difference for LPs and inclinometers at maximum displacement for joint #2

<table>
<thead>
<tr>
<th>Relative to J2_LP2 (Degree)</th>
<th>Relative to J2_LP3 (Degree)</th>
<th>LPs Average (Degree)</th>
<th>Inclinometer (Degree)</th>
<th>δI-LP2 (Degree)</th>
<th>δI-LP2 (%) Difference</th>
<th>δI-LP3 (Degree)</th>
<th>δI-LP3 (%) Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.211</td>
<td>1.250</td>
<td>1.231</td>
<td>1.126</td>
<td>0.085</td>
<td>7.55</td>
<td>0.124</td>
<td>11.01</td>
</tr>
</tbody>
</table>

Table 2-8 Rotation measurement difference for LPs and inclinometers at maximum displacement for joint #3

<table>
<thead>
<tr>
<th>Relative to J3_LP2 (Degree)</th>
<th>Relative to J3_LP3 (Degree)</th>
<th>LPs Average (Degree)</th>
<th>Inclinometer (Degree)</th>
<th>δI-LP2 (Degree)</th>
<th>δI-LP2 (%) Difference</th>
<th>δI-LP3 (Degree)</th>
<th>δI-LP3 (%) Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.610</td>
<td>0.658</td>
<td>0.634</td>
<td>0.585</td>
<td>0.025</td>
<td>4.27</td>
<td>0.073</td>
<td>12.47</td>
</tr>
</tbody>
</table>

Degrees of rotation from LPs were calculated based on averaged displacement readings at the peak vertical uplift displacement (i.e 25 mm). Based on the three comparisons, it is concluded that the inclinometers and LPs produced accurate readings.

Axial translations were calculated for Joint #1 and #3. From geometry, axial translation is calculated using the formula in Equation 2-3.

\[ b \left( \frac{LP_1 - LP_2}{h} \right) + LP_2 \quad \text{(Equation 2-3)} \]

This formula calculates the axial translation in millimetres, where \( b \) is the vertical distance, in millimetres, from haunch LP to pipe centre. \( H \) is the vertical distance from haunch LP to crown LP, in millimetres, 75.38 mm for this instrumentation scheme. Vertical distance between pipe
centre and haunch LP is approximately 25.11 mm. Figure 2-22 to Figure 2-24 depict the calculated axial compression or extension from the second articulation test. \( J(1,2,3)_{LP2} \) represents axial compression as measured by the haunch LP2 in the joint of interest and respectively for \( J(1,2,3)_{LP3} \) for axial translation measured by haunch LP3.

![Graph of Axial Extension](image)

**Figure 2-22** Axial (horizontal) extension in joint #1 for articulation test #2
Figure 2-23 Axial (horizontal) extension in joint #2 for articulation test #2

Figure 2-24 Axial (horizontal) extension in joint #3 for articulation test #2
From the calculations and plots, Joints #1 and #2 experienced axial compression while Joint #3 experienced axial extension. Axial compression/extension did not occur until actuator stroke had reached 5 mm (i.e at the end of the 1st lift step). Magnitude of axial extension was smaller in Joint #3 in the first lift step compared to that of Joint #1. Both joints recorded linearity in axial translation during lifting and lowering of the pipe assembly; this indicated that both Joint #1 and Joint #3 experienced a uniform lifting/lowering motion during the articulation test. Both joints recorded approximately the same magnitude of axial movement in terms of absolute values, Table 2-9 summarizes axial translation for all three joints at maximum vertical displacement. Negative sign denotes axial compression and positive sign denotes axial extension.

Table 2-9 Axial extension of joints at maximum vertical displacement (measurement is millimetres)

<table>
<thead>
<tr>
<th>LP(s)</th>
<th>Joint #1</th>
<th>Joint #2</th>
<th>Joint #3</th>
</tr>
</thead>
<tbody>
<tr>
<td>LP2</td>
<td>-0.687</td>
<td>-0.523</td>
<td>+0.596</td>
</tr>
<tr>
<td>LP3</td>
<td>-0.669</td>
<td>-0.479</td>
<td>+0.556</td>
</tr>
</tbody>
</table>

In Joint #2 the haunch LPs reported complex movements in terms of axial translation. In reference to Equation 2-3 for displacement, the haunch LPs displayed movements that were different than those reported in haunch LPs inside adjacent joints (i.e Joint#1 and #3). Assuming displacement readings from haunch LPs in Joint #1 and #3 implied only vertical movements of the joints; a probable cause of the complex displacement and axial translation readings in Joint #2 was that a combination of both lateral and vertical movements were experienced by Joint #2 during lifting by the hydraulic actuator.
2.6.3. Strain Gauge Readings from Articulation Test

A total of twenty-four strain gauges were installed on the four-pipe assembly; each pipe had a total six strain gauges. Three strain gauges were installed at the crown position longitudinally along the surface of pipe, and another three were deployed along the invert. Based on geometry and simple beam analysis, the expected strain for each pipe under its self-weight is approximately 6.1 με. The calculation was based on the assumption of clay’s Young’s modulus of 70 GPa. Self-weight of pipe assumed to be 400 N. Strain gauges were labelled as follows:

Table 2-10 Strain gauge labels

<table>
<thead>
<tr>
<th>Crown Position</th>
<th>Test Pipe 1 (TP1)</th>
<th>Test Pipe 2 (TP2)</th>
<th>Test Pipe 3 (TP3)</th>
<th>Test Pipe (TP4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bell</td>
<td>TP1_SG1_CROW</td>
<td>TP2_SG1_CROW</td>
<td>TP3_SG1_CROW</td>
<td>TP4_SG1_CROW</td>
</tr>
<tr>
<td>Mid-Section</td>
<td>TP1_SG2_CROW</td>
<td>TP2_SG2_CROW</td>
<td>TP3_SG2_CROW</td>
<td>TP4_SG2_CROW</td>
</tr>
<tr>
<td>Spigot</td>
<td>TP1_SG3_CROW</td>
<td>TP2_SG3_CROW</td>
<td>TP3_SG3_CROW</td>
<td>TP4_SG3_CROW</td>
</tr>
<tr>
<td>Bell</td>
<td>TP1_SG1_INVER</td>
<td>TP2_SG1_INVER</td>
<td>TP3_SG1_INVER</td>
<td>TP4_SG1_INVER</td>
</tr>
<tr>
<td>Mid-Section</td>
<td>TP1_SG2_INVER</td>
<td>TP2_SG2_INVER</td>
<td>TP3_SG2_INVER</td>
<td>TP4_SG2_INVER</td>
</tr>
<tr>
<td>Spigot</td>
<td>TP1_SG3_INVER</td>
<td>TP2_SG3_INVER</td>
<td>TP3_SG3_INVER</td>
<td>TP4_SG3_INVER</td>
</tr>
</tbody>
</table>
TP2 and TP3 were most critical due to their connection at Joint #2, which was the point where vertical displacement was imposed during the tests. Figure 2-25 to Figure 2-28 present the strain readings during Articulation Test #2.

Figure 2-25 Strain readings on test pipe #1 (TP#1)
Figure 2-26 Strain readings on test pipe #2 (TP#2)

Figure 2-27 Strain readings for test pipe #3 (TP#3)
Three strain gauges were omitted from this presentation due to large accumulating strains and off-scale readings. These omitted strain gauges are: TP1_SG3_INVERT, TP2_SG3_INVERT, and TP3_SG2_INVERT. In general, strains were within expected values (i.e. 6 με as calculated from self-weight of pipe and simple beam analysis). Strain gauges in Test Pipe #2, in particular, performed within expectation. It recorded an increase in strain as the pipe segment that was lifted by the actuator, and reached a peak value at maximum displacement. Strains reduced and returned to original values when the pipe segment was lowered back to its original position. The majority of the strain gauges reported reasonable strain values; in Chapter 4, they will be utilized to measure longitudinal strains.

### 2.6.4. Total Station Reading during the Articulation Tests

A total station recorded changes in positions for reflective prisms mounted on all three joints. These positions were measured relative to Cartesian axes (i.e. X, Y, and Z). A benchmark was taken prior to commencement of the articulation tests. Figure 2-29 displays all three axes for

![Figure 2-28 Strain readings for test pipe #4 (TP#4)](image.png)
reference purposes. Figures 2-30 and 2-31 then outline the relative position (to the benchmark) on the X-Y plane during the two articulation tests. All measurements are in millimetres.

Figure 2-29 Reference Cartesian axes for articulation tests
Figure 2-30 Total station readings (relative positions) during articulation test #1

Figure 2-31 Total station readings (relative positions) during articulation test #2
Lifting the pipe assembly by actuator occurred in Steps 1 to 5, and the lowering of the assembly occurred in Steps 6 and 7. From Figures 2-30 and 2-31, it appeared that Joint #2 experienced the highest degree of lateral movement. Readers should bear in mind that the entire pipe assembly was lifted vertically by the actuator on Joint #2 only. Since Joint #2 could not be placed perfectly under the actuator and there was some slack in the ratchet straps, some degree of lateral movement was induced during lifting. Lateral movement was at its greatest particularly in Steps 4 and 5, where Step 5 corresponded to the maximum vertical displacement. From the first articulation test, Joint #2 settled slightly southeast from its original position – approximately 1.5 mm south and 3.0 mm east from the origin. In contrast, Joint #1 experienced very little lateral movement and the movement was mostly confined to an east-west orientation. However, Joint #3 showed a high degree of lateral movement for both orientations. The magnitude of lateral movement for Joint #3 was equal if not greater than those experienced by Joint #2; it increased drastically at Step 4 and reached its maximum during Steps 5 and 6. At the end of both tests, Joint #3 returned to its original position in the north-south orientation, however, it was displaced 6.5 mm to the East after Articulation Test #1 and 2.5 mm to the East after Articulation Test #2. In conceptual terms, however, it might not be reasonable to expect the same or greater degree magnitude of lateral movements in the subsequent full-scale burial tests given the confining pressure provided by 0.6 metres and 1.2 metres of soil cover. Based on results from the LPs in this articulation test, it seemed that the LP provided the lateral movements that occurred on these joints in subsequent tests, if any.

2.7. Articulation Test Summary

In summary, the articulation test reported in this chapter revealed that the inclinometers fitted within the pipes performed within expectations based on geometrical calculations. The linear
potentiometers’ (LPs) performances were verified against inclinometer readings in terms of joint rotation. The difference in angles as obtained by inclinometers and LPs was between 0.009 and 0.124 degree. For the subsequent buried pipe experiments, the desired degree of rotation is approximately 1°. The expected discrepancy is between 0.9 to 12%. Table 2-11 summarizes the measurement discrepancy between haunch LPs in Test #2 at maximum displacement.

Table 2-11 Degree discrepancy measured between haunch LPs in joints from test #2 at maximum vertical displacement

<table>
<thead>
<tr>
<th>Joint</th>
<th>δ_{LP2-LP3} (Degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint #1</td>
<td>0.022</td>
</tr>
<tr>
<td>Joint #2</td>
<td>0.039</td>
</tr>
<tr>
<td>Joint #3</td>
<td>0.048</td>
</tr>
</tbody>
</table>

The small discrepancy measured between LPs is an indication that the LPs performed within design intent. There is a percentage difference between the average degree measured by LPs and degree measured by inclinometer during maximum displacement as Table 2-12 summarizes:

Table 2-12 Percentage difference of degree measured during test #2

<table>
<thead>
<tr>
<th>Joint</th>
<th>Percentage (%) Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint #1</td>
<td>1 - \left( \frac{0.625}{0.624} \right) = -0.2%</td>
</tr>
<tr>
<td>Joint #2</td>
<td>1 - \left( \frac{1.126}{1.231} \right) = 8.5%</td>
</tr>
<tr>
<td>Joint #3</td>
<td>1 - \left( \frac{0.585}{0.634} \right) = 7.7%</td>
</tr>
</tbody>
</table>

There seems to have been re-alignment and lateral movement of all joints during the articulation test as confirmed by the total station readings. In addition, data noise from the inclinometers could explain the difference between rotations obtained from LPs and inclinometers readings.
Strain gauges also performed within calculated expectation from simple beam analysis. Three strain gauges were determined to have malfunctioned due to accumulating strain and offscale readings. The strain gauges would be utilized in subsequent full-scale tests to determine bending moments induced by differential ground movement.

Total station readings confirmed lateral movements in both East-West and North-South orientations for all three joints during both articulation tests. Joint #2 experienced the greatest degree of lateral movements as expected given that it was the only lifting point of the pipe assembly. Joint #1 had experienced the least amount of lateral movements; in contrast, Joint #3 underwent the same magnitude, if not a greater degree of lateral movements compared to Joint #2. It was clear that there was a realignment of all joints after lifting of the pipe assembly. Based on total station readings, it appeared that haunch LPs in Joint #2 underwent a combination of lateral movement (in both orientations) and vertical displacement.
2.8. References


Chapter 3 Test Chamber for Simulating Normal Ground Faults

3.1. Introduction

This chapter provides details of the test chamber used to simulate normal ground faults. It covers the chamber design, its construction, and results from the preliminary test conducted using olivine test sand. The test is used to assess the chamber’s ability to simulate the soil movements induced by a normal ground fault imposed on the rigid base, prior to conduct of the buried pipe tests reported in Chapter 4 where buried pipe responses are evaluated under those differential ground movements. The patterns of free-field displacements are subsequently determined from the digital photographs of the test soil and PIV (Particle Image Velocimetry), work performed by Ni (2015), who contributed significantly to the design and construction of the test chamber.

3.2 Chamber Design

Figure 3-1 shows the vertical cross-section of the new test chamber developed to conduct experiments examining the nature and impact of differential ground movements associated with imposition of a normal ground fault. The chamber is approximately 7.3 metres long and 1.8 metres wide, and was designed to house the soil and any pipe assembly buried within. The floor of the test chamber consists of two parts, a moveable portion and a stationary portion. Both portions are 3.65 metres long. The chamber is orientated longitudinally in the North-South orientation within the Eastern half of the large test pit within the GeoEngineering Laboratory located at the West Campus of Queen’s University. Concrete blocks were used to construct the East wall of the chamber. These concrete blocks were of three lengths: 2.7 m (9-feet), 1.8 m (6 feet), and 0.9 m (3 feet). All were 0.6 m (2-feet) in height. Two glass panels (i.e. windows) designed by Burnett (2014) were placed on top of each other. Each panel is 2.44 m (8 feet) long.
and 1.22 m (4 feet) high. The glass panels allowed visual observation of free-field soil displacements and the motion of the Northern end of the moving floor to be captured by digital cameras. Figure 3-1 also presents the configuration of precast concrete blocks used to make up the East wall. In that figure, #1 denotes blocks 3-feet long, #2 for blocks 6-feet long, and #3 for blocks 9-feet long. Each block was 2-feet high and 3-feet wide.

Figure 3-1 Concrete block configuration for the eastern (primary) wall

An additional row of interlocking concrete blocks was stacked longitudinally against the primary concrete block wall to ensure adequate factors of safety against sliding and rotation (toppling) failure under the influence of soil loading. Figure 3-2 depicts the concrete block arrangement for the secondary wall.
For factor of safety against sliding, the author employed Coloumb’s method for active and at-rest conditions. Soil pressure was analyzed using the equivalent fluid method. Table 3-1 summarized assumptions made for these calculations.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Value (and Corresponding Unit)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{\text{Concrete}}$</td>
<td>24 kN/m$^3$</td>
<td>Unit Weight of Concrete</td>
</tr>
<tr>
<td>$\gamma_{\text{Sand}}$</td>
<td>15 kN/m$^3$</td>
<td>Unit Weight of Sand</td>
</tr>
<tr>
<td>$\phi_{\text{Sand}}$</td>
<td>35$^\circ$</td>
<td>Internal Angle of Friction (Sand)</td>
</tr>
<tr>
<td>$\mu$</td>
<td>0.30</td>
<td>Coefficient of Static Friction for Concrete</td>
</tr>
</tbody>
</table>

For calculation of factor of safety against rotation, the at-rest condition was considered based on the design assumptions summarized above. Calculation for active condition against rotation was unnecessary because coefficient of lateral earth pressure ($K_o$) is greater than the coefficient of active lateral earth pressure ($K_a$). Table 3-2 summarized factors of safety against sliding and rotation for the at-rest conditions for the wall sections adjacent to both the moveable and stationary sections of the test chamber (shown subsequently in Figure 3-3). The wall adjacent to the stationary part of the test chamber also has to support a layer of compacted granular soil.
Table 3-2 Factors of safety against sliding and rotation failures

<table>
<thead>
<tr>
<th></th>
<th>F.O.S against Sliding Failure (Moveable)</th>
<th>F.O.S against Sliding Failure (Stationary)</th>
<th>F.O.S against Rotation Failure (Moveable)</th>
<th>F.O.S against Rotation Failure (Stationary)</th>
</tr>
</thead>
<tbody>
<tr>
<td>At-Rest</td>
<td>1.27</td>
<td>1.41</td>
<td>14.93</td>
<td>8.72</td>
</tr>
</tbody>
</table>

The factors of safety calculations are located in Appendix C of this thesis. These reveal that the second row of concrete blocks was necessary to guard against sliding. Additional restraint against sliding was also added to support the wall adjacent to the stationary part of the floor; a steel angle was placed along the base, and anchored to the floor of the pit using drop-in anchors employing eight, ½” Grade 5 steel bolts. Two six-foot (0.6 m) long concrete blocks with a total height of 4 feet (1.2 m) were placed width-wise across the chamber to separate the moveable and stationary parts of the chamber (seen later in Figure 3-3). An access point was left immediately to the South of those blocks to allow person entry to inspect and adjust all four screw jacks. The access point provided is directly below the windows.

Figure 3-3 displays the conceptual moving floor design assembly (along with locations of the screw jacks).
As shown in Figure 3-3, the stationary (Northern) portion of the chamber is filled with poorly-graded Gravel with Sand soil from base level to height of 1.2 m (4 feet) (i.e. up to the top position of the moveable floor). After covering this with geotextile, the remaining height was filled with olivine test sand (i.e. the section of the test chamber above the initial level of the moving floor). Total depth of the olivine sand in both stationary and moveable portion was 1.8 m.

Four screw jacks were utilized to hold the moveable floor. Each jack had a load capacity of 15 metric tons in compression. With four screw jacks, the total load capacity of this design was approximately 60 metric tons. The four screw jacks were worm gear screw jacks distributed by KAMOTION Group Incorporated and manufactured by Merkur. They had a 300 mm vertical stroke. Two of these screw jacks were previously used for a research project in the GeoEngineering Laboratory, and two identical jacks were purchased, the screw jacks' data were from Becerril García (2015).
Each pair of screw jacks was spaced approximately 2.13 m apart centre to centre; the Southern jacks were approximately 0.63 m from edge of southern wall and the Northern jacks were approximately 0.59 m from the concrete blocks which separated the chamber. The screw jacks were designed to allow for approximately 17.5 cm of vertical displacement. For the subsequent experiments involving burial of a pipe assembly reported in Chapter 4 of this thesis, the magnitude of vertical displacement of the moveable floor of from 25 to 50 mm (1 to 2 inches) was envisaged.

A steel structure was designed to support the moving floor, as shown in Figure 3-4, with wooden sheets on top to hold the test sand. Hollow Structural Sections (HSS) with square shape were chosen as the frame material, with the primary supporting frame made up of four longitudinal beams welded to four transverse beams (the primary frame supports a secondary steel frame and the wooden sheets). The top of each screw jack was located within a pair of the transverse beams in the primary frame and held in place using steel pins (as shown in a subsequent figure). Beam analysis was conducted to determine dimensions of the HSS beams required, with design to ensure that bending stress at the extreme fibres does not exceed 340 MPa (the steel used is classified as CSA G40.20 grade 350W, Class C as specified by Canadian Institute of Steel Construction). Beams were also designed so that bending deflections are less than 1 mm. The design calculations are shown in Appendix C of this thesis. For the transverse beams within the primary frame, the design required an HSS beam with cross-sectional width x cross-sectional height x wall thickness of 89x89x9.5 mm, while the longitudinal members were fabricated from HSS of 64x64x6.4 mm. Locations of the two pairs of transverse beams used within the primary frame were placed to optimize bending moments in the frame which reduces the possibility for a
certain portion of the frame to experience higher degree of bending moment than another portion.

The secondary frame was composed of thirteen lighter weight transverse beams, with longitudinal end-beams added to tie the secondary frame together. Based on a factor of safety of two against bending moment and shearing, the lighter-weight transverse beams and the end-beams were fabricated from HSS members of 51x51x3.2 mm. In Figure 3-4, yellow members denote the primary supporting frame; and black members denote the secondary frame.

Figure 3-4 Steel frame with side beams (dimensions are in metres)

Figure 3-5 and 3-6 show the steel floor frame after fabrication.
The upper frame composed of the thirteen lighter-weight transverse beams and the end beams was bolted along the two pairs of longitudinal beams within the primary frame at a centre to centre spacing of approximately 0.3 m (1 foot).

When designing the test chamber, a decision was required regarding the chamber width. Cheuk et al. (2005) and Liu et al., (2012) examined soil deformations resulting from uplift of buried pipes, as shown in Figure 3-7, where the lateral zone of influence depends on the outer diameter.
of the test pipe and the angle of dilation of the soil. For the case of a buried rigid pipe spanning across a normal ground fault (the problem being examined in this thesis), uplift movements of the pipe are expected relative to the soil in the zone immediately adjacent to the position of the ground fault, but over the side where the moveable floor is being lowered. It is this location where uplift controls the zone of influence and is the primary consideration when choosing a width sufficient to avoid boundary effects.

![Zone of Influence Diagram](image)

Where:
- **OD** – Outer Pipe Diameter
- **ψ** - Dilation Angle (Degree)

Figure 3-7 Zone of influence based on soil dilation angle during uplift (after Cheuk et al., 2005)

The dilation angle of the olivine test soil lies between 22 to 25°. This meant that the lateral zone of influence would be approximately 1.2 m to 1.4 m for a burial depth of 1.2 m. A trench width of 1.8 m was therefore chosen, so it exceeded the expected zone of influence and limits boundary effects during differential ground movement. This dimension was also chosen because it was a multiple of the 0.3m (1 ft) which was the horizontal dimension associated with the precast concrete blocks used in the laboratory (as per the earlier discussion). Figure 3-8 shows how the selected trench width looks relative to the O.D. (outer diameter) of the clay pipe bell to be tested,
as well as the ratio of burial depth to the O.D. of the bell for one of the test conditions examined in Chapter 4. The trench width to diameter ratio for the 274.6 mm diameter test pipe discussed in Chapter 4 is approximately 6.5.

Figure 3-8 Width and burial depth to pipe ratio (when used for clay pipe testing in Chapter 4)

### 3.3 Chamber Construction

The first stage of the construction involved the Western wall of the test chamber. This wall was built parallel to the removable reinforced concrete retaining wall used to divide the testing area in the GeoEngineering Laboratory into two sections (Cholewa, 2009). The laboratory’s retaining wall is of cantilever type, with feet extending into the Eastern half of the lab’s large test pit. The Western wall of the chamber was fabricated using sheets of plywood supported by wood studs anchored to the retaining wall. Wooden studs of 2 by 4 inches were used to create a wooden frame and to ensure the Western wall was vertical. The studs were spaced 0.3 m (1 foot) apart.
Figure 3-9 displays the wooden frame constructed over the ends of the concrete feet of the retaining wall located immediately to the West.

![Completed wooden frame (west side of chamber)](image)

Figure 3-9 Completed wooden frame (west side of chamber) situated on the eastern portion of the reinforced concrete cantilevered retaining wall.

The central section of the laboratory’s concrete retaining wall featured a gap for access during research on trenchless construction operations. This gap had an approximate length (North-South) of 30 inches (76.2 cm) and a depth (East-West) of 24 inches (60.9 cm). Beams composed of three 2 by 6 inch wooden studs were used to span across this gap, Figure 3-10 displays those wooden beam reinforcements.
After completion of the wooden frame, sheets of plywood 8 feet in length and 4 feet in width were nailed in place on the Eastern side of the Western wall. Plywood with thickness of \( \frac{3}{4} \) inch (1.90 cm) was used. Figure 3-11 displays installation of that plywood.
Masonite boards were adhered to the plywood using polyurethane and nailed into place to create a smooth surface, since the plywood surface could have introduced a significant degree of friction impeding floor and soil movement. Figure 3-12 shows the installation of the Masonite boards. Figure 3-13 shows the Western wall completely covered with Masonite.
The second stage of chamber construction involved placement of pre-fabricated concrete blocks to act as supports for the Eastern wall. The blocks were moved into position by using an
overhead crane. The Eastern wall consisted of two walls designated as primary and secondary. The first set of blocks to form the primary wall adjacent to both stationary and moveable portions of the test floor was carefully levelled (vertically and horizontally) with wooden shims to provide an even surface for a layer of 1/2" (1.27 cm) thick plywood. A secondary set of blocks was subsequently placed on the Eastern side of the primary layer to ensure adequate factors of safety against sliding and toppling.

The Southern wall was constructed using the same methods as the Western wall. A wooden frame was constructed from 2 by 4 inches and 2 by 6 inches wooden studs supported against the South concrete wall of the test chamber with 2 by 4 inches studs. The offset of this wooden frame relative to the concrete wall was approximately 0.61 m (2 feet). Figure 3-14 displays the Southern wall frame. The South wall was then completed using ¾ inch (1.90 cm) thick plywood sheets and Masonite boards. Figure 3-15 displays sections of all three completed walls (Western, Eastern, and Southern).
The steel frame was lowered into the chamber using the overhead crane, and placed on wooden columns for temporary support. For safety, the steel frame was also held by the overhead crane.
prior to installation of the four screw jacks. Figure 3-16 displays the frame placement on the temporary supports prior to jack installation.

Figure 3-16 Steel frame placement prior to jacks' installation

The screw jacks were fixed onto the concrete floor by the use of drop-in concrete anchors, which were connected to their baseplates using 9/16” bolts. Figure 3-17 displays the baseplate connection to the concrete floor.
The baseplates were shimmed with 1/8 inch (3.17 mm) to 5/16 inch (7.93 mm) thick steel plates to level the surface and to ensure all jacks were vertical relative to the floor. A total of four jacks were utilized to hold the steel frame, a pair towards the South end, and a pair towards the North end. Each pair of jacks was connected by a steel driveshaft. A steel pin connected the top of each jack to the steel frame. Figures 3-18 and 3-19 display all four jacks connected by drive shafts, along with connections to the steel frame.
Two wooden cradles were constructed to support the end of the two drive-shafts coming from the screw jacks as displayed in Figure 3-20. Given the location of the Southern jacks, a 4 inch
(10.16 cm) diameter hole had to be cored through the concrete blocks at the base of the Eastern wall to accommodate the drive-shaft.

![Diagram of a cored hole, drive-shaft, and wooden cradle]

Figure 3-20 Drive-shaft for southern jacks

The drive-shaft for the Northern jacks was directly below the window frame and held into place by its wooden cradle.

Two glass panels with their frames (windows) were attached to the concrete blocks of the Eastern wall by 1/2 inch, Grade 5 steel bolts. Each glass frame was held in place by a total of four steel bolts (two bolts at each side). The two frames were also held together by 1/2 inch, Grade 5 steel bolts. Further support was provided by steel plates installed into the concrete blocks by drop-in anchors, which buttressed the window frames to prevent outward movement (displacement in the Eastern direction). Figures 3-21 and 3-22 show the glass frames (windows) after installation.
The two concrete blocks 1.8 m (6 feet) wide, 0.9 m (3 feet) deep, and 0.6 m (2 feet) high, were then placed in the middle of the chamber to separate it into its two portions: stationary and moveable. The top block was specially cast so it had a flat upper surface instead of the male part.
of the shear key that these precast retaining wall blocks otherwise have (seen clearly on the upper surface of the blocks shown in Figure 3-25). At this point plywood sheets ½ inch (1.27 cm) thick were laid down onto the steel frame forming the movable floor and bolted into place with 7/16 inch (1.11 cm) carriage bolts, each 5 inches (12.70 cm) long. Figure 3-22 shows the completed wooden surface over the steel frame.

Figure 3-22 Plywood installed over the steel frame of the moveable floor

To ensure no olivine sand can leak around the edges of the moveable floor, flexible plastic sheets were placed along the edges, and thin aluminum bars were used to hold these sealing layers in place as shown in Figure 3-23.
At the interface between the moving floor and the concrete block (the interface between the stationary and moveable parts of the floor), a double-flap sealing system was used. One piece of plastic was placed at the top of concrete block while another piece was placed below it. The concept is to create mobility at this interface, which allows the two layers of polymer sheet to glide on top of each other while maintaining a seal against sand leakage. Figure 3-24 displays this sand sealing system and Figure 3-25 depicts the system against the concrete block/moveable floor interface.
Figure 3-24 Double sand sealing system at concrete interface

Figure 3-25 Concrete block/moving floor Interface (this image shows the moveable floor before it was lifted up to match the level of the upper concrete block)
Geotextile was placed around the corners of the moveable floor to provide additional sealant against sand leakage. Figure 3-26 displays the completed plastic sealing skirt around the moveable floor.

![Figure 3-26 Plastic sealant for moveable floor](image)

Friction treatment was applied on all four walls of the chamber. The system used was that developed by Tognon et al. (1999) and employed in many buried pipe experiments (e.g. Brachman et al., 2000; Dhar and Moore, 2002). It featured use of special lubricant between two polyethylene sheets, and limited the friction angle to approximately 4 degrees.

Olivine sand was used for all experiments reported in this thesis undertaken with the test chamber simulating normal ground faults. Based on the dimensions of the chamber, approximately 24 m$^3$ of sand was required. The majority of the olivine test sand was initially wet after being stored for a period of time, and had to be air-dried by exposing it to the open environment prior to testing. The wet sand was thinly spread over an area with rakes and hand shovels for approximately 24 hours or longer (depending on reduction of moisture). Figure 3-27 displays that preparation of the olivine sand.
After sufficient reduction of moisture was achieved, the sand was placed into bags for temporary storage and protection from further exposure.

The next stage involved backfilling of poorly graded Gravel with Sand (GP-SP) into the stationary portion of the chamber below the level of the moveable floor as shown in Figure 3-3. The height of this backfill is 4 feet (121.92 cm), and it was compacted using a Wacker Neuson gas vibratory plate compactor with two passes around the entire area. To prevent moisture infiltration into the olivine sand above and mixing of the soil materials placed above and below this level (i.e. olivine sand and GP-SP, respectively), two pieces of geotextile sandwiching a sheet of polyethylene were used to provide a moisture and physical barrier. Figure 3-28 displays the poorly-graded Gravel with Sand prior to geotextile placement.
3.4 Soil Placement and Compaction

Olivine sand was placed in 6 inch (0.15 m) thick lifts using a soil hopper, which maintained constant height and soil flow during placement. Three density cups of known masses and
volumes were placed in pre-determined locations along the chamber to measure sand density of each lift placement. Figure 3-29 presents the locations of the density cups.

![Density cup placements within the chamber (plan view) (measurements are in centimetres)](image)

Olivine sand was compacted around four edges of the chamber by hand-tamping using a steel plate measuring 10 inches by 10 inches (25.4 cm by 25.4 cm), and dropped from a height of approximately 6 inches (15.2 cm). The entire perimeter of the chamber was hand-tamped once and the inner area was compacted once using a gas-powered vibrating plate. Overall, dry densities during the entire backfilling process ranged between 1490 and 1740 g/cm³, the variations of densities are due to locations density cup placement (for investigative purposes) and different thicknesses of lifts (which will be explained later in this section). The dry density of the olivine sand at the centreline of chamber was found to vary between 1650 and 1720 g/cm³ using the compaction procedure described above. Density cups were also placed at the edge of the
West and East walls during several lifts to investigate dry density achieved in those locations; values were typically between 1610 and 1650 g/cm³. Figure 3-30 summarizes the dry densities measured using all three density cups for each lift placement (a total of twelve lifts were placed).

![Figure 3-30 Summary of dry densities for soil only burial test](image)

Near the end of backfilling at Lift #11, there was a shortage of olivine sand to reach required depth of burial at the stationary part of the chamber; an extra bag of sand was added to stationary end (North portion) of the chamber to maintain a level ground surface prior to testing. This caused Lift #12 to be slightly thicker (than the standard 0.15 m thick lift) and hence a lowered dry density.
3.5 Test Result and Discussion

To monitor vertical displacement of the steel frame during the experiment, one stringpot was attached to each pair of screw jacks. The screw jacks were manually lowered using crank wheels. Both stringpots were zeroed and calibrated prior to commencement of the test. The soil-only test reported here was conducted on May 21 2015.

A total of four cameras (all were the Rebel T4i model manufactured by Canon) were utilized to capture photos at a 10 second interval for PIV (particle image velocimetry) and photogrammetry analyses. Cameras #1 and #2 were positioned above the soil surface and were used for photogrammetric analysis, the objective being evaluation of the soil surface deformations resulting from the vertical displacement of the moveable floor. Cameras #3 and #4 were positioned facing the window frame to capture soil deformations beneath the surface as seen through the windows using PIV analysis, Camera #3 recording the top window, and Camera #4 the bottom window. Figure 3-31 presents the camera locations in plan view.
The steel frame was lowered vertically in 5 mm intervals (steps) with one minute between each interval for stabilization readings of both cameras and stringpots. A total of 122 mm of vertical floor displacement was applied in this test. Figure 3-32 presents readings from both stringpots during the test.
Both stringpots confirm that the steel frame had displaced evenly (the North jack displacements are hidden perfectly under the South jack displacements) and manually operated jacks can produce the controlled frame descent needed for the subsequent experiments involving the buried pipe assembly.

In addition, no leakage of olivine sand was observed during the entire duration of this soil-only test. This confirms that the plastic skirt around the steel frame performed as designed. Figure 3-33 shows the final view of the soil through the upper window, and the final geometry of the soil surface concerns that the normal-fault test chamber is able to produce differential ground movement and the associated soil deformations.
Figure 3-33 Deformed soil from frame articulation test (top window frame)

3.6 Summary

A test chamber 1.8 m wide, 7.3 m long, and 1.8 m deep was constructed to study how differential soil settlements result as a normal ground fault is imposed at the base, and how those deformations influence pipes buried within the soil. The test chamber featured a moveable floor along the Southern half, and a stationary floor along the Northern half. The moveable floor was supported by four screw jacks, which were used to control its vertical settlement. The test chamber was built with smooth vertical walls, and it was fitted with a friction treatment system to ensure angle of friction was approximately 4 degrees. Chamber width was chosen to ensure upward movement of segments of pipe in the subsequent buried pipe tests had influence length less than the distance between the sidewalls (so pipe uplift does not influence soil movements at the chamber walls).
A soil-only test was performed to examine the effectiveness of the sealing system used to prevent the dry olivine test sand from escaping below or besides the moveable floor. Based on the result from that test, the chamber performed within design expectations and intent. No leakage of olivine sand was observed during the entirety of this test. The chamber was demonstrated to be capable of producing differential ground movements associated with a total of 122 mm of vertical floor displacement. Based on compaction results from density cup measurements, the use of the vibrating plate compactor produced olivine densities between 1650 and 1720 g/cm³. Table 3-3 summarizes the mean and one standard deviation of soil densities, densities from Lift #12 are omitted given that it is a thicker lift (as an extra bag of sand was added due to shortage of sand at Lift #11 at stationary end of chamber) than standard lift of 0.15 m.

Table 3-3 Mean and standard deviation of olivine sand densities

<table>
<thead>
<tr>
<th>Statistics</th>
<th>Olivine Sand Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>1677</td>
</tr>
<tr>
<td>Standard Deviation (σ)</td>
<td>44</td>
</tr>
</tbody>
</table>

This range of olivine sand densities was subsequently considered to be the “dense” state for the experiments reported in Chapter 4, where tests on the vitrified clay pipe assembly under normal ground fault would be reported.
3.7 References


Lapos, B.M. (2004). Laboratory study of static pipe bursting: three-dimensional ground displacements and pull force during installation, and subsequent response of HDPE replacement pipes under surcharge loading, MSc thesis, Department of Civil Engineering, Queen’s University, Kingston, Ontario, Canada.


Chapter Four Field Scale Testing of Jointed Clay Pipe Subjected to Normal Ground Fault

4.1 Introduction

Chapter Four of this thesis provides information regarding the field scale testing of the jointed pipe assembly at both 1.2 m and 0.6 m burial depths. The tests involve the entire clay pipe assembly discussed previously in Chapter Two of this thesis. The pipe assembly was placed into the test chamber introduced in Chapter Three, and subjected to normal vertical differential ground movement. The test preparation, olivine sand details and test results from three sets of instruments (linear potentiometers, inclinometers, and strain gauges) form the core of this chapter.

4.2 Test Design and Procedures for 1.2 m Burial Depth

The first buried pipe test involved a burial depth of 1.2 m as measured from the springline to surface as shown in Figure 4-1. In subsequent discussion, this first configuration is regarded as the deep burial test.
Friction treatment was applied on all four walls of the chamber using the same procedure described in Chapter Three. The system used was that developed by Tognon et al. (1999) and has since been employed in many buried pipe experiments (e.g. Brachman et al., 2000; Dhar and Moore, 2002). It features use of special lubricant between two polyethylene sheets, and limits friction angle to approximately 4 degrees.

To monitor movement of the polyethylene sheet in contact with the soil relative to the sheet in contact with the walls of the test chamber, depths between 0.6 and 1.5 m were outlined with adhesive tape at intervals of 0.3 m on the Eastern, Western, and Southern walls. The rationale for this was to provide visual confirmation and measurement of the relative movement of the two layers as the olivine sand mobilizes the friction treatment during backfilling and testing. Figure 4-2 displays depth intervals outlined on the Eastern wall of the moveable portion of the test.
chamber (that part where the floor is suspended on screw jacks, and which is lowered during
testing to simulate a normal ground fault).

![Figure 4-2](image)

**Figure 4-2 Depth markers on eastern wall (moveable portion of test chamber)**

Prior to backfilling of the olivine sand, reference elevation measurement (or shots) were taken on
the four corners of the steel frame of the test chamber and two more shots on the top concrete
block, which separated the moveable and stationary portions of the test chamber. Table 4-1
summarizes the elevation shots and benchmark elevation in metres. The benchmark was set at a
point on the Southern concrete wall in the GeoEngineering Laboratory. Elevation measurements were taken during backfilling to control lift thickness.

### Table 4-1 Elevation measurements (pipe at 1.2 m burial depth) prior to backfilling

<table>
<thead>
<tr>
<th>Elevation Shots</th>
<th>Elevation (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Benchmark</td>
<td>1.740</td>
</tr>
<tr>
<td>South-East Corner (Moveable Portion)</td>
<td>3.580</td>
</tr>
<tr>
<td>South-West Corner (Moveable Portion)</td>
<td>3.585</td>
</tr>
<tr>
<td>North-West Corner (Moveable Portion)</td>
<td>3.583</td>
</tr>
<tr>
<td>North-East Corner (Moveable Portion)</td>
<td>3.580</td>
</tr>
<tr>
<td>North-West Edge (Concrete Separator Block)</td>
<td>3.567</td>
</tr>
<tr>
<td>Mid-Span (Concrete Separator Block)</td>
<td>3.564</td>
</tr>
</tbody>
</table>

Backfilling of the olivine was conducted in the same manner as outlined in Chapter Three. The sand was placed in 6 inch (0.15 m) thick lifts using a soil hopper, which maintained constant height and soil flow during placement. The sand was compacted around the perimeter once using hand-tamping with a steel plate measuring 10 inches by 10 inches (25.4 cm by 25.4 cm), dropped from a height of approximately 6 inches (15.2 cm). The inner area was then compacted once using the gas-powered Wacker Neuson vibratory plate compactor. Again, three density cups of known masses and volumes were placed in each layer at pre-determined locations. In addition to density cups, soil samples were collected from at various lifts for the determination of the olivine sand’s moisture content. Figure 4-3 presents the locations of the three density cups.
The entire pipe assembly was placed in Lift #5 with an average soil base elevation of 3.055 m relative to the top of the moveable floor at the base of the test chamber. The three density cups were placed near the springline of the pipe assembly on the west side of the test chamber for Lift #5. The overhead crane lowered the pipe assembly longitudinally along the centreline of the test chamber. The soil surface was leveled manually using rakes. A trench for the pipe assembly was then manually excavated and surface elevations verified by the laser level prior to placement of pipe assembly. Figure 4-4 displays the pipe assembly placed into the test chamber prior to it being backfilled.
In addition, elevation shots were taken at the crown of each joint and mid-span of each pipe segment prior to backfilling. This was to determine any net vertical displacement of these points after the pipe assembly was exhumed post-testing. Figure 4-5 summarizes the elevation shots of the pipe assembly pre-testing. All points lay at or between elevations of 2.819 and 2.875 m.

Lifts #5 and #6 were hand-tamped four times using the steel plate across the entire chamber area, with overlap to ensure adequate soil compaction (the vibrating plate was not employed since...
these layers contained the pipe). An external stringpot was connected to the crown of Joint #4 to monitor horizontal motion of the pipe assembly in the axial direction, if any. The stringpot was mounted within a wooden box to protect it against soil pressure during backfilling and testing, as shown Figure 4-6 the stringpot location after post-test exhumation.

![Figure 4-6 External stringpot for measuring axial (horizontal) movement at Joint #4](image)

Two cameras manufactured by Canon (Rebel T4i model) were used to photograph both top and bottom window frames during testing (for subsequent PIV analysis to be performed by Ni, 2015, as mentioned previously in Chapter Three). Figure 4-7 depicts the placement of the two digital cameras. Camera #1 was used to capture images of the top window frame and Camera #2 for the bottom window frame. Both cameras were programmed to capture photographs at 10 second intervals during the time period when the ground fault movements were being simulated.
4.3 Soil Placement and Details for 1.2 m Burial Depth

This section of Chapter Four is reserved for further discussion of the olivine sand placement and additional information regarding the olivine sand density after placement. A total of twelve soil lifts were placed into the chamber to achieve a total soil height of 1.8 m. Olivine sand densities were measured at every lift using three density cups of known masses and volumes. Figure 4-8 summarizes dry soil density measured during the backfilling process.
Overall, the dry densities after compaction ranged between 1500 and 1750 g/cm³. Lifts #5 and #6 were hand-tamped using the steel plate twice across the entire test area, and this may have contributed to the lower density observed in Lift #5 (the location of the pipe assembly). All other lifts were compacted using the procedures mentioned in Chapter Three. Tables 4-2 and 4-3 summarize dry density statistics for all twelve lifts.

Table 4-2 Dry densities statistics (excluding lift #5 and #6) for test at 1.2 m burial depth

<table>
<thead>
<tr>
<th>Statistics</th>
<th>Dry Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>1677</td>
</tr>
<tr>
<td>Standard Deviation (σ)</td>
<td>46</td>
</tr>
</tbody>
</table>

Table 4-3 Dry densities statistics (lift #5 and lift #6 only) for test at 1.2 m burial depth

<table>
<thead>
<tr>
<th>Statistics</th>
<th>Dry Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>1580</td>
</tr>
<tr>
<td>Standard Deviation (σ)</td>
<td>48</td>
</tr>
</tbody>
</table>
Moisture content of the olivine sand was investigated during backfilling. A total of nine soil samples were collected during backfilling, with sets of three samples collected from Layers #5, #8 and #10. Sand samples were oven-dried over a 24 hour period at approximately 110 °C. Table 4-4 summarizes the moisture content data that resulted.

Table 4-4 Corresponding moisture content to lift placement for test at 1.2 m burial depth

<table>
<thead>
<tr>
<th>Lift #</th>
<th>Average Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.51</td>
</tr>
<tr>
<td>8</td>
<td>0.57</td>
</tr>
<tr>
<td>10</td>
<td>0.76</td>
</tr>
</tbody>
</table>

One soil sample was lost in Lift #5 and subsequently not including in the calculation for moisture content. As shown in the table above, the moisture content of the olivine sand was less than 1%. The average moisture content from all samples was 0.63% with a standard deviation of 0.14%. For the purposes of placement, compaction and subsequent testing, the sand behaved as if it were essentially dry.

During soil placement, a laser level was utilized to record soil lift elevations. Five reference points were employed along the length of the test chamber. Figure 4-9 outlines the locations of the reference points.
Figure 4-9 Reference points for lift elevation shots (for test at 1.2 m burial depth) (measurements are in centimetres) (plan view)

Figure 4-10 summarizes the burial depths of nine of the lifts during backfilling.
The average base elevation is 3.582 m. A small thin layer of olivine was placed after Lift #12 to complete the final surface elevation of 1.772 m which resulted in a total depth of approximately 1.81 m of soil.

### 4.4 1.2 m Burial Depth Test

The 1.2 m burial test was conducted on June 3, 2015. The moveable floor was lowered at 2 mm intervals with a minute left in between for readings to stabilize. The frame was lowered in a total of fifteen stages, which equated to approximately 30 mm of total vertical displacement. The test duration was 50 minutes. Figure 4-11 displays the frame displacement as recorded by the two stringpots monitoring the floor position (one attached to the northern transverse beams, another onto the southern transverse beams as defined in Chapter 3).
Figure 4-11 Frame displacement during test at 1.2 m burial depth (measurements are in millimetres)

It appeared that the manually operated screw jacks produced an even and coordinated vertical displacement during the test, with the North and South displacement readings mapped one on top of the other.

4.5 Inclinometer Results from 1.2 m Burial Test

All inclinometers functioned throughout the test, with three of the four inclinometers recording very little rotation during the experiment. The only significant rotation was reported by I2_Inclinometer which was situated in the bell portion of Joint #2, though small rotations were also observed in the inclinometer within the bell of Joint #3. Figure 4-12 displays the raw data as reported by all four inclinometers.
Joint rotations were calculated by taking the difference between the inclinometers readings as described in Chapter Two of this thesis. Figure 4-13 summarizes the joint rotations for this particular test. This calculation is based on the assumption that each pipe segment rotated as a rigid body, with all slope (inclination) changes being accommodated by the joints. This assumption is examined in further detail in a subsequent section where rotations based on linear potentiometer data are discussed.
The inclinometers infer that Joints #2 and #3 experienced rotations of -1.15° and 0.99°, respectively. In contrast, Joint #1 experienced rotation of approximately 0.20°. The data shown in Figure 4-13 had been smoothed (averaged) and normalized. Based on the geometry discussed in Chapter Two, the joint rotation during the articulation test was approximately 1 degree at the centre joint (i.e. Joint #2) for a vertical displacement of 25 mm. In this test, however, the pipe assembly required approximately 30 mm of vertical displacement to produce a joint rotation of 1 degree at the centre joint.

**4.6 Linear Potentiometer Results from 1.2 m Burial Test**

A total of nine linear potentiometers (LP) were used for this test, the same configuration that was used in the articulation test discussed in Chapter Two. The LPs were used to measure joint movements so that rotation and horizontal extension/compression of the joints can be evaluated. During the 1.2 m burial test, none of the LPs reported malfunction by the data acquisition system. Figure 4-14 to Figure 4-16 display the joint rotations as calculated from LPs'
measurements. Joint rotation was measured relative to both haunch LPs (i.e Joint#1_LP2/3) using the approach introduced in Chapter 2.

Figure 4-14 Joint #1 rotation during buried pipe test at 1.2 m burial depth

Figure 4-15 Joint #2 rotation during buried pipe test at 1.2 m burial depth
Based on results from the LPs, Joint #1 experienced the least amount of rotation as expected (given that the joint was in the stationary portion of the test chamber and confined by the static soil mass). The joint rotations as obtained by J1_LP2 and J1_LP3 were 0.06 degree and 0.07 degree, respectively, by the end of this test. In Joint #2 and Joint #3, the LPs reported a nearly equal and opposite degree of joint rotation at the end of the test. LPs in Joint #2 measured joint rotations of approximately -0.95 degree and -0.99 degree from J2_LP2 and J2_LP3, respectively. Similarly, in Joint #3, J3_LP2 measured a joint rotation of 0.98 degree and J3_LP3 measured 1.02 degree. Based on the reported LP data and their behaviours, it is indicative that all LPs performed as intended during this 1.2 m burial test.

From the LP data, axial horizontal extension of all three joints was calculated as discussed in Chapter Two. Figure 4-17 to Figure 4-19 present horizontal extension longitudinally along the pipe across all three joints, based on LP calculations presented in Chapter 2. Positive value is defined as axial (horizontal) compression, whereas, negative value is defined as axial...
(horizontal) extension. Extensions were calculated separately for each haunch LP. All measurements are in millimetres.

Figure 4-17 Axial (horizontal) extension of joint #1 for test at 1.2 m burial depth

Figure 4-18 Axial (horizontal) extension of joint #2 for test at 1.2 m burial depth
Joint #1 reported very little axial horizontal extension for the duration of the test. Given the location of Joint #1 over the stationary portion of the test chamber floor, the miniscule axial extension appears to be reasonable. However, it appeared that the extension experienced by Joint #1 almost fully recovered at the end of the test; where the Joint #1 initially experienced axial compression during vertical displacement. But at 1680 second, axial extension occurred. At the end, J1_LP2 measured a horizontal extension of less than 0.01 mm. In contrast, measurements from J1_LP3 indicated an almost complete recovery of axial extension once vertical displacement ceased.

Figures 4-18 and 4-19 show a net average axial compression in Joint #2 of approximately 0.85 mm and a net average axial extension in Joint #3 of approximately 1.17 mm. Joint behaviours of these three joints and their interpretations will be discussed subsequently in this chapter.
4.7 Comparisons of Inclinometers and Linear Potentiometers for 1.2 m Burial Test

In this section, comparisons of joints rotations as obtained by both inclinometers and linear potentiometer are presented. Figure 4-20 to Figure 4-22 display joint rotations as obtained by both sets instruments. Angle is in absolute values for comparison purposes.

Figure 4-20 Comparison of inclinometers and LPs in joint #1 for 1.2 m burial test
For both Joints #2 and #3, the inclinometers readings were similar to values calculated from measurements with LPs. Table 4-5 summarizes the differences (in absolute values) in readings between inclinometers and LPs for Joint #2 and Joint #3 at the conclusion of the test.
Table 4-5 Inclinometers and LPs degree comparison for joint #2 and #3 at the end of 1.2 m burial depth test (at 3000 second)

<table>
<thead>
<tr>
<th>Joint</th>
<th>Difference ([J(x)_{\text{LP2}}]) between Inclinometers</th>
<th>% Difference Relative to Inclinometers for ([J(x)_{\text{LP2}}])</th>
<th>Difference ([J(x)_{\text{LP3}}]) between Inclinometers</th>
<th>% Difference Relative to Inclinometers for ([J(x)_{\text{LP3}}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.180</td>
<td>15.8</td>
<td>0.140</td>
<td>12.3</td>
</tr>
<tr>
<td>3</td>
<td>0.019</td>
<td>1.9</td>
<td>0.060</td>
<td>6.2</td>
</tr>
</tbody>
</table>

Joint #2 reported a higher difference between the two sets of instruments than Joint #3. For Joint #1, the discrepancy can be due to bending of the pipe barrel between joints during the ground displacement – this producing a difference in the inclinometer and LP measurements. This discrepancy will be discussed further subsequently in this chapter.

In general, the LP and inclinometer measurements appeared to be in close agreement – supporting their further analysis and interpretation.

4.8 Strain Measurements for 1.2 m Burial Test

As mentioned in Chapter Two, a total of twenty-four strain gauges were installed onto the pipe assembly for strain measurements. On each test pipe, three strain gauges were located on the crown and another three on the invert; all were evenly spaced longitudinally along the pipe (they are placed at the \(\frac{1}{4}\), \(\frac{1}{2}\), and \(\frac{3}{4}\) positions).

Strain gauges were utilized in this 1.2 m burial test to record strains associated with longitudinal bending as experienced by the pipe assembly. Of a total of twenty-four gauges, three malfunctioned during the test (this was indicated by offscale readings), those designated TP1_SG1_CROWN, TP2_SG3_INVERT, and TP3_SG1_INVERT. The remaining twenty-one strain gauges were responsive and performed consistently during the test. Figure 4-23 to 4-26
present strain measurement (in microstrain) for all four test pipes during the entire 1.2 m burial test. All measurements had been smoothed (averaged within each interval of 5 time steps) and normalized (by subtracting strain readings with the averaged strain of the first 100 seconds of the test) for presentation.

Figure 4-23 Strain measurements for test at 1.2 m burial depth (TP1)
Figure 4-24 Strain measurements for test at 1.2 m burial depth (TP2)

Figure 4-25 Strain measurements for test at 1.2 m burial depth (TP3)
Test Pipes #1 and #4 reported lower strains as compared to Test Pipes #2 and #3. This observation supports the fact that both Test Pipe #1 and #4 were furthest from the fault, and the differential ground movement (DGM) would have a smaller effect (in terms of strain) on these pipes. In each case, strain gauges on the top and bottom of the pipe provide values that are almost equal in magnitude but opposite in sign (tensile versus compressive). This observation supports the conclusion that the strain gauges operated effectively during the test, and that each pipe experienced longitudinal bending due to DGM with little axial force (or axial force insufficient to generate significant axial strains). For example, Test Pipe #4 exhibited ‘hogging’ curvature, with tensile strain along the crown and compressive strain along the invert. Figure 4-27 summarizes the final strain measurements of the entire pipe assembly at the conclusion of the test (at 3000 second).
Figure 4-27 Total incremental strain measurements at the end of the 1.2 m burial depth test

As Figure 4-27 has shown, Test Pipes #2 and #3 at mid-span recorded the highest degree of strain and values that were equal and opposite. In addition, the strain values at mid-span of Test Pipe #3 are slightly higher than those at the mid-span of Test Pipe #2. A possible mechanism is that as the pipe assembly was subjected to DGM, Test Pipe #3 was "dragged" along by the soil mass and pushed downwards to the fault hence the slightly higher strain measurements. Figure 4-28 idealizes this mechanism.

Figure 4-28 Idealization of pipe drag during differential ground movement
4.9  Post 1.2 m Burial Depth Test Details

This section presents post-test results of the test at 1.2 m burial depth. First, the displacement of the friction treatment was measured post-test. Friction treatment displacements of both stationary and moveable portions of the test chamber were measured. Figure 4-29 displays the relative movement of the friction treatment observed post-test at the 1.5 m depth mark on the moveable East wall.

![Figure 4-29 Post-test (1.2 m burial depth) friction treatment displacement (at moveable, East Wall, 1.5 m mark)](image)

~3.8 cm displacement

In general, the relative displacement observed across the friction treatment was about 3.5 to 4.0 cm on the moveable portion, and about 0.6 to 0.8 cm on the stationary portion of the test.
chamber. Tables 4-6 and 4-7 summarize further details of the friction treatment displacements around the test chamber.

Table 4-6 Friction treatment displacements on moveable portion (1.2 m burial depth test)

<table>
<thead>
<tr>
<th>Reference Markers (m)</th>
<th>East Wall (cm)</th>
<th>West Wall (cm)</th>
<th>South Wall (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>3.8</td>
<td>4.0</td>
<td>3.7</td>
</tr>
<tr>
<td>1.2</td>
<td>3.6</td>
<td>4.0</td>
<td>3.7</td>
</tr>
<tr>
<td>0.9</td>
<td>3.8</td>
<td>3.8</td>
<td>3.6</td>
</tr>
<tr>
<td>0.6</td>
<td>3.8</td>
<td>3.8</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Table 4-7 Friction treatment displacements on stationary portion (1.2 m burial depth test)

<table>
<thead>
<tr>
<th>Reference Markers (m)</th>
<th>East Wall (cm)</th>
<th>West Wall (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>0.6</td>
<td>0.8</td>
</tr>
<tr>
<td>1.2</td>
<td>0.8</td>
<td>0.6</td>
</tr>
<tr>
<td>0.9</td>
<td>0.6</td>
<td>0.7</td>
</tr>
<tr>
<td>0.6</td>
<td>0.5</td>
<td>0.6</td>
</tr>
</tbody>
</table>

There is consistent evidence that the friction treatment performed successfully during testing, especially around the moveable portion of the test chamber. The friction treatment also indicates that the soil mass in the stationary portion of the test chamber moved much less as the soil compressed under its self-weight and as a result of compaction. In addition, this displacement of the friction treatment provides information about the soil compaction (the friction treatment on the stationary side displaced by about 7 mm), whereas, a displacement of approximately 38 mm was recorded on the friction treatment around the moveable portion. The difference between
these two values, a difference of 31 mm – is essentially the magnitude of downward translation of the test chamber floor applied by the screwjacks.

Second, elevations taken at certain locations of the pipe assembly before and after testing were compared to determine the magnitude of vertical displacements. Elevation measurements were taken at the crown of each joint and mid-span of every pipe segment. Figure 4-30 displays elevation measurements after testing (all elevations are in metres). Figure 4-31 summarizes the changes in elevations of the pipe assembly during the 1.2 m burial test.

![Figure 4-30 Elevations of pipe assembly post-test at 1.2 m burial depth (elevations in metres)](image)

![Figure 4-31 Elevation changes of pipe assembly for test at 1.2 m burial depth (elevations in metres)](image)

Test Pipes #3 and #4 situated in the moveable portion of test chamber recorded higher changes of elevation than those in Test Pipes #1 and #2 (situated in the stationary portion of test chamber). In Joints #3 and #4, the changes in elevation were 31 and 38 mm, respectively. The change is slightly greater than the total vertical displacement of the test, which was 30 mm of vertical displacement. The extra pipe deformation of approximately 8 mm in Joint #4 likely partly
corresponds to the settlements that resulted as soil was placed over the pipe during backfilling, and is essentially the same motion observed across the friction treatment on the walls around the stationary part of the test chamber.

Lastly, the external springpot registered minimal horizontal movement of the pipe assembly. Figure 4-32 presents the small deformations recorded for the end of the pipe relative to the soil during the 1.2 m burial depth test movements, so small that they were within the resolution of the string potentiometer being employed.

Figure 4-32 External horizontal displacement of the end of the pipe assembly (test at 1.2 m burial depth) (measurement is in millimetres)

As Figure 4-32 shown there was only a net horizontal movement between 0.04 and 0.06 mm, and it is concluded that the end of the pipe assembly remained almost stationary during the entire 1.2 m burial depth test.
4.10 Test Design and Procedures for 0.6 m Burial Depth

For this field scale test, the pipe assembly was buried 0.6 m (the distance from its springline to the soil surface). For discussion purpose of this thesis; it is regarded as the shallow burial test. As shown in Figure 4-33, the pipe assembly was buried at a depth of 0.6 m relative to its springline with a bedding thickness of 1.2 m.

Figure 4-33 Vertical section showing geometry of the first buried pipe test at 0.6 m burial depth; (width and burial depth relative to pipe diameter are also shown.)

Again, friction treatment was used on all four walls (Western, Eastern, Southern, and Northern) in the same manner as for the deep burial test. In addition, adhesive tapes were applied to Eastern, Western and Southern walls to outline depths between 0.6 and 1.5 m at an interval of 0.3m.
Prior to backfilling of the olivine sand, reference elevation shots were taken along the steel frame of the test chamber and two more shots on top of the concrete block, which separated the moveable and stationary portions of the test chamber. Table 4-8 summarizes the elevation shots recorded prior to backfilling for lift thickness control.

**Table 4-8 Elevation measurements for test at 0.6 m burial depth prior to backfilling**

<table>
<thead>
<tr>
<th>Elevation Shots</th>
<th>Elevations (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Benchmark</td>
<td>1.740</td>
</tr>
<tr>
<td>South-West Corner (Moveable)</td>
<td>3.576</td>
</tr>
<tr>
<td>Mid-span of Steel Frame (West Wall)</td>
<td>3.578</td>
</tr>
<tr>
<td>North-West Corner (Moveable)</td>
<td>3.574</td>
</tr>
<tr>
<td>North-West Edge (Concrete Separator Block)</td>
<td>3.567</td>
</tr>
<tr>
<td>Mid-Span (Concrete Separator Block)</td>
<td>3.564</td>
</tr>
</tbody>
</table>

Olivine sand was placed in the same manner as the deep burial test. Again, three density cups were placed along the centreline of the test chamber at the same locations as detailed by Figure 4-3. The entire pipe assembly was placed at Lift #9 with an average trench base elevation of 2.447 m. The soil surface was leveled manually using ranks; the trench for the pipe assembly was manually excavated and surface elevations verified by the laser level prior to placement of pipe assembly. An overhead crane was used to lower the pipe assembly into the test chamber. Figure 4-34 shows the pipe assembly at the 2.447 m elevation mark prior of it being backfilled.
Figure 4-34 Pipe assembly in the test chamber for test at burial depth of 0.6 m (prior to backfilling)

Elevation shots of the pipe assembly after placement were documented for subsequent post-test analysis. Figure 4-35 summarizes the elevation shots of the pipe assembly prior to backfilling.

Figure 4-35 Pipe assembly elevations prior to backfilling (0.6 m burial test)

As in the 1.2 m burial test, an external stringpot was attached to the bell (the springline of J4) of the pipe assembly to monitor any external axial movement during DGM. Two identical cameras
were also set up in the same configuration as depicted in Figure 4-7. The cameras were manufactured by Canon (Rebel T4i model); the cameras were time-lapsed to photograph the two glass frames at 10-second intervals for the purpose of PIV analysis to be performed by Ni (2015) as detailed in Section 4.2 of this chapter.

4.11 Soil Placement and Details for 0.6 m Burial Depth

This section presents information regarding the olivine sand and its placement for the shallow burial test. A total of twelve soil lifts were placed for this shallow burial experiment, for a total height of 1.8 m. Olivine sand dry densities were measured using density cups of known masses and volumes. Figure 4-36 summarizes the dry densities of the olivine sand for all twelve lifts at the time of placement.

![Figure 4-36 Summary of dry densities for test at 0.6 m burial depth](image)

Figure 4-36 Summary of dry densities for test at 0.6 m burial depth
The pipe assembly was placed at Lift #9, hand tamping using a steel plate was utilized for Lift #9 and Lift #10; the entire area of the test chamber was hand-tamped twice for each lift. This was done to avoid damaging the instruments in/on the pipe assembly. Overall, the dry densities of all the soil lifts ranged between 1530 and 1756 g/cm³. In Tables 4-9 and 4-10 summarizes the statistics of the dry densities of the soil lifts.

Table 4-9 Dry density statistics (excluding Lifts #9 and #10) for test at 0.6 m burial depth

<table>
<thead>
<tr>
<th>Statistics</th>
<th>Dry Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>1693</td>
</tr>
<tr>
<td>Standard Deviation (σ)</td>
<td>38</td>
</tr>
</tbody>
</table>

Table 4-10 Dry density statistics (Lifts #9 and #10 only) for test at 0.6 m burial depth

<table>
<thead>
<tr>
<th>Statistics</th>
<th>Dry Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>1570</td>
</tr>
<tr>
<td>Standard Deviation (σ)</td>
<td>24</td>
</tr>
</tbody>
</table>

Moisture content of the olivine sand was investigated during backfilling. A total of three soil samples were collected during backfilling; the soil samples were taken from Lift #2. Samples were oven dried over a period of 24 hours at a temperature of approximately 110 °C. Average moisture content is 0.33% with a standard deviation of 0.08%.

With the average moisture content of less than 1%; for the purposes of placement, compaction and subsequent testing, the sand behaved as if it were essentially dry.

During soil placement, a laser level was utilized to record soil lift elevations. Five reference points along the longitudinal length of the test chamber were recorded, as depicted in Figure 4-9. Figure 4-37 displays the burial depths of all twelve lifts.
The average base elevation is 3.575 m. A small thin layer of olivine was placed after Lift #12 to complete the final average surface elevation of 1.766 m which equates to approximately a total soil height of 1.809 m.

### 4.12 Test at 0.6 m Burial Depth

The shallow burial test was conducted on June 23, 2015. The moveable floor was lowered at 2 mm intervals with a minute for readings to stabilize; the frame was lowered in fifteen stages, which equated to approximately 30 mm of vertical displacement. The test duration was 50 minutes. Figure 4-38 displays the frame displacement as recorded by the two stringpots (one attached to the north transverse beams, another onto the south transverse beams).
Again, it appears that the manually operated screw jacks produced an even and coordinated vertical displacement during the test.

### 4.13 Inclinometer Results from 0.6 m Burial Test

Though all inclinometers performed during the test, three of the four inclinometers recorded very little rotation. The only significant rotation was reported by I3 Inclinometer which was situated in the bell portion of Joint #3. Figure 4-39 displays the raw data as reported by all four inclinometers.
Figure 4-39 Inclinometer data from test at 0.6 m burial depth

Figure 4-40 the total inclinations of all three joints are summarized.

Figure 4-40 Joints rotations for test at 0.6 m burial depth (in degrees)

As shown in Figure 4-40, there was very little joint rotation in Joint #1 (situated over the stationary end of the test chamber). However, the inclinometers indicated that significant
rotations occurred across Joint #2 and #3. Joint rotations during the test were approximately -
1.30 and 1.23 degree for Joint #2 and Joint #3, respectively. In Joint #1, only 0.11 degree was
reported during this shallow test. The rotation angles in Figure 4-40 had been normalized and
smooth-averaged. Based on inclinometer readings, it required approximately 26 mm of vertical
displacement of the moving floor to produce approximately 1.0 degree of joint rotation in both
Joint #2 and Joint #3.

4.14 Linear Potentiometer Results from 0.6 m Burial Test
The LP instrumentation scheme used for the shallow burial test is the same as the deep burial
test. No error from the LPs was reported by the data acquisition system during this shallow test.
In Figure 4-41 to Figure 4-43 display the joint rotations as calculated from the LP measurements.
Joint rotations were calculated using both haunch LPs (i.e J1_LP2/3).

Figure 4-41 Joint #1 rotation for test at 0.6 m burial depth
Based on results from the LPs, Joint #1 experienced the least amount of rotation as expected; given the joint was located over the stationary portion of the test chamber and confined by the soil mass. The joint rotations as calculated using J1_LP2 and J1_LP3 were 0.09 degree and 0.07
degree, respectively, by the end of this test. LPs in Joint #2 measured a joint rotation of approximately -1.18 degree from both J2_LP2 and J2_LP3. Similarly, in Joint #3, use of the J3_LP2 measurement gave a joint rotation of 1.03 degrees and J3_LP3 provided a rotation of 1.07 degrees. These results indicate that all LPs performed as intended during this 0.6 m shallow burial test.

From the LP data, axial horizontal translation (either extension or compression) of all three joints was calculated as discussed previously in Chapter 2. In Figure 4-44 to Figure 4-46 present horizontal extension longitudinally along the pipe for all three joints based on the calculation from the LP readings. Positive value is defined as axial (horizontal) compression, whereas, negative value is defined as axial (horizontal) extension. Axial extensions were calculated using displacements measured with both haunch LPs separately. All measurements are in millimetres.

Figure 4-44 Axial (horizontal) extension of joint #1 for test at 0.6 m burial depth
From Figure 4-44, there was no net axial horizontal extension in Joint #1. In Joint #2 as shown in Figure 4-45, there was a net axial horizontal compression of approximately 0.124 mm; it is significantly less than the joint experienced in the 1.2 m burial test. In contrast, compared to the
deep burial test, the axial compression behaviour in Joint #2 from this test is not as smooth and lock-stepped as shown in Figure 4-18. Similarly, LPs in Joint #3 measured a small amount of axial extension, approximately 0.48 mm of net axial extension as obtained by J3_LP2 and 0.51 mm as obtained by J3_LP3.

In Joint #1, there was a complete recovery of the axial extension at the conclusion of the shallow burial test. It appears that gasket contributed to this recovery of axial extension. From Figures 4-45 and 4-46, similar axial extension behaviours are exhibited as the 1.2 m burial test; where there is small initial compressional/extensional axial extension behaviour but once a threshold extension has been attained, the opposite axial extension motion becomes dominant. This behaviour will be further analyzed and discussed in a subsequent section of this chapter.

4.15 Comparisons of Inclinometer and Linear Potentiometers for 0.6 m Burial Test

In this section, comparisons of joints rotations as obtained using both inclinometers and linear potentiometer from this shallow test are presented. In Figure 4-47 to Figure 4-49 display joint rotations as obtained using both sets of instruments. Angle is in absolute degrees for comparison purposes.
Figure 4-47 Comparison of inclinometers and LPs in joint #1 for test at 0.6 m burial depth

Figure 4-48 Comparison of inclinometers and LPs in joint #2 for test at 0.6 m burial depth
For Joint #1, it appears that joint rotations as obtained by both inclinometer and LPs are in close agreement. In contrast, Figures 4-48 and 4-49 have shown slight discrepancies in joint rotations at the conclusion of this shallow burial test. In particular, in Joint #2 the rotation measurements by inclinometers and LPs are not exactly aligned; this phenomenon also appeared in the 1.2 m burial test in Joint #2 as shown in Figure 4-21. In Joint #3 from this shallow burial test, the rotations obtained by both sets of instruments are in close to each other until 2000 seconds where a small divergence occurs. Table 4-11 summarizes the differences (in absolute values) in readings between inclinometers and LPs for all joints at the conclusion of the test.
Table 4-11 Inclinometers and LPs degree comparison for all joints at the end of test at 0.6 m burial (at 3000 second)

<table>
<thead>
<tr>
<th>Joint</th>
<th>Difference $[J(x)_{LP2}]$ between Inclinometers</th>
<th>% Difference Relative to Inclinometers for $[J(x)_{LP2}]$</th>
<th>Difference $[J(x)_{LP3}]$ between Inclinometers</th>
<th>% Difference Relative to Inclinometers for $[J(x)_{LP3}]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.044</td>
<td>52.6</td>
<td>0.061</td>
<td>73.7</td>
</tr>
<tr>
<td>2</td>
<td>0.123</td>
<td>9.5</td>
<td>0.116</td>
<td>8.9</td>
</tr>
<tr>
<td>3</td>
<td>0.179</td>
<td>17.9</td>
<td>0.160</td>
<td>13.0</td>
</tr>
</tbody>
</table>

In terms of magnitude of differences in joint rotation measurement, Joint #1 reported the smallest difference. In contrast, Joint #3 reported the greatest in terms of magnitude of measurement difference. In Joint #2 the percentage difference in rotation measurement is below 10%. Therefore, it appears LPs and inclinometers measurements for Joint #2 provided reliable measurements. As for Joint #1 and Joint #3, it seems additional physical mechanisms caused discrepancies in rotation measurements. These physical mechanisms will be under discussion later in this chapter.

**4.16 Strain Measurements for 0.6 m Burial Test**

Strain gauges were utilized in this 0.6 m burial test to record strains as experienced by the pipe assembly. Of a total of twenty-four gauges, one malfunctioned during the test (it became offscale). The gauge that malfunctioned was TP3_SG1_INVERT. The remaining twenty-three strain gauges were responsive and functional during the test. In Figure 4-50 to Figure 4-53 present strain measurement (in microstrain) for all four test pipes during the entire 0.6 m burial test. All measurements had been smoothed (averaged within each 5 time steps) and normalized (by subtracting strain readings with the averaged strain of the first 100 seconds of the test).
Figure 4-50 Strain measurements during test at 0.6 m burial (TP1)

Figure 4-51 Strain measurements during test at 0.6 m burial depth (TP2)
As expected Test Pipe #1 experienced the least amount of strain since this pipe was buried in the stationary part of the test chamber. Strain values from TP4_SG1_CROWN fluctuated as compared to other strain gauges on Test Pipe #4 (and have thus been removed from Figure 4-53).
Overall, strain values at crown and invert of all test pipes are approximately equal but opposite to each other. This is a reasonable strain response for pipes exposed to bending.

Figure 4-54 summarizes the strain values at the end of the shallow test (at 3000 seconds).
Figure 4-54 Strain measurements at end of 0.6 m burial test
As Figure 4-54 shows, strain gauge TP4_SG1_CROWN reported a questionable reading at the end of this test. It is worth noting that Test Pipe #3 experienced higher strain readings than Test Pipe #2. It might be due to Test Pipe #3 being “pulled” downwards by the DGM during the shallow test. Test Pipe #1 experienced the least amount of strain followed by Test Pipe #4; this seems reasonable as both pipes are at the far ends of the test chamber and would experience the least amount of DGM. It is interesting to observe that the strains at crown and invert are in opposite pairs (the crown reading is almost equal and opposite to the invert reading) for Test Pipe #2 and Test Pipe #3.

4.17 Post 0.6 m Burial Depth Test Details

This section presents post-test results of the 0.6 m shallow burial test. First, the displacement of the friction treatment was measured post-test. Friction treatment displacements of both stationary and moveable portions of the test chamber were measured. Figure 4-55 displays the mobilized friction treatment post 0.6 m burial test.
Overall, the friction treatment was displaced between 3.6 and 4.0 cm on all three walls (West, East and South) around the moveable portion. For the stationary portion of the test chamber, the friction treatment displaced between 1.0 and 2.5 cm on the East and West walls. This could be due to a shortage of friction treatment lubricant during treatment at the stationary portion, and there was a brief separation of friction treatment layers during backfilling due to strong air circulation.

It appears that the friction treatment performed just as well as in the deep burial test around the moveable portion of the test chamber. The friction treatment also indicates that the soil mass in the stationary portion of the test chamber moved less as the soil compressed under its self-weight and as a result of compaction. However, the stationary side was displaced at a greater magnitude in this test than the 1.2 m burial depth test. This might be due to lack of friction treatment
lubricant (near the end of the friction treatment process) and a brief separation of friction treatment layers during backfilling which could have contributed to an extra amount of displacement recorded at the end of this 0.6 m burial depth test.

Second, elevations taken at certain locations of the pipe assembly prior and post-tests were compared to determine the magnitude of final vertical displacements. Elevation measurements were taken at crown of each joint and the mid-span of every pipe segment.

Figures 4-56 and 4-57 summarize the pipe assembly elevations post-test and the changes in elevation relative to pre-test positions (all elevations are in metres).

![Figure 4-56 Elevations of pipe assembly post-test at 0.6 m burial depth (elevations in metres)](image)

![Figure 4-57 Elevation changes of pipe assembly for test at 0.6 m burial depth (elevations in metres)](image)

Shown in Figure 4-57, there is a significant movement of Joint #3, Test Pipe #4, and Joint #4. For Joint #3, the amount of vertical displacement is greater than the DGM imposed during this experiment (i.e 30 mm), partly or wholly the result of the vertical soil compression that occurs during compaction, that adds to the movement resulting from floor translation. In contrast, Test Pipes #1 and #2 likely experienced downward movements due to soil compression during
backfilling, but thereafter remained stationary during floor translation, since both pipes then remain stationary.

Lastly, the external axial (horizontal) extension of the pipe assembly during the 0.6 m burial test is presented in Figure 4-58.

![Figure 4-58 External horizontal displacement of pipe assembly (0.6 m burial depth)](image)

(measurement is in millimetres)

These displacements of this end of the pipe assembly fluctuated between -0.06 and 0.07 mm, and were again appear to have been within the measurement ability of the string potentiometer, so are subsequently considered to be negligible.

### 4.18 Analysis and Discussion of the 1.2 m and 0.6 m Burial Depth Tests

This section presents the discussion and analysis of the two burial tests. This discussion includes: differences between rotations for different burial depths, relationships between joint rotation and vertical ground displacement, relationships between joint axial (horizontal) extension and
vertical ground displacement, and the likely physical mechanisms which contributed to
differences in joint rotation measurements obtained from the LPs and the inclinometers.

First, the differences between joint rotations as obtained by LPs from deep and shallow burial
tests are analyzed. In Figure 4-59 to Figure 4-61 present the LPs measurements of joint rotations
from both burials tests.

Figure 4-59 Joint #1 rotations (as obtained by LPs) from tests at 1.2 and 0.6 m burial depths
As shown in the three previous figures, Joint #2 reported the highest degree in difference of rotation measurements, followed by Joint #3 and Joint #1 (with the smallest difference). It appears the burial depths have some moderate influence on joint rotations, particularly in Joint #2 (above where the fault is located) than the remaining two joints (where they are situated away...
from the normal ground fault). Table 4-12 summarizes the statistical difference in joint rotations (in absolute values and in degrees) over the course of both buried pipe tests.

Table 4-12 Joint rotation differences statistics

<table>
<thead>
<tr>
<th>Joint</th>
<th>Average (in Degrees)</th>
<th>Standard Deviation (σ) (in Degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint #1</td>
<td>0.02</td>
<td>0.01</td>
</tr>
<tr>
<td>Joint #2</td>
<td>0.09</td>
<td>0.03</td>
</tr>
<tr>
<td>Joint #3</td>
<td>0.05</td>
<td>0.02</td>
</tr>
</tbody>
</table>

From the statistics, Joint #2 displays the greatest amount of difference in joint rotations as expected, where DGM would have the highest degree of influence on this joint. In addition, the average differences in joint rotations at Joint #1 and Joint #3 are smaller. Therefore, it would be reasonable to assume different burial depths contribute little, if any, additional joint rotation for a certain amount of DGM at joints situated far from the fault.

Now, the relationship between DGM and joint rotation is explored. Rotations from Joints #2 and #3 are plotted against vertical ground displacement in Figures 4-62 and 4-63, respectively. Both burial tests were subjected to a DGM of 30 mm, which facilitates this comparison. Joint #1 is excluded from this analysis due to the small amount of rotations experienced in both tests as shown earlier in Figures 4-14 and 4-41. In Figures 4-62 and 4-63 display the joints’ rotations versus vertical ground displacement (in millimetres).
Figure 4-62 Joint #2 rotations versus vertical ground displacement from tests at 1.2 and 0.6 m burial depths

Figure 4-63 Joint #3 rotations versus vertical ground displacement from tests at 1.2 and 0.6 m burial depths

$y = -0.0339x + 0.0593$  \hspace{1cm} R² = 0.9953

$y = -0.0367x + 0.0392$  \hspace{1cm} R² = 0.9986

$y = 0.035x - 0.0657$  \hspace{1cm} R² = 0.9942

$y = 0.036x - 0.0337$  \hspace{1cm} R² = 0.9976
From Figures 4-62 and 4-63, it is apparent that joint rotations develop almost linearly with vertical ground displacement. However, the response is somewhat non-linear, as observed in both joints at both burial depths (except for Joint #2 at burial depth of 0.6 m). Based on visual inspection, it appears that the non-linearity occurs initially and ceases at a vertical displacement between 2.0 and 2.1 mm. This initially non-linear behaviour may be a result of the characteristics of the friction treatment or the moving floor system used in this testing chamber (perhaps it takes a small amount of movement to overcome the initial resistance of the moving floor and/or the wall friction). However, this will not be examined further in this thesis, and will be left for investigation in subsequent studies.

Linear trend lines have been fitted for analysis of joint rotation versus vertical displacement, and the R-squared values range between 0.994 and 0.998. This indicates that there is a close approximation of the linear trend lines to the data. The slopes of the trend lines in Figure 4-62 indicate that there is a small influence from vertical differential ground movements on joint rotation for the pipe buried at 1.2 m and 0.6 m burial depths in Joint #2. The two trend lines in Figure 4-62 could be utilized in further studies for approximate predictions of the vitrified clay pipe assembly’s Joint #2 rotation when it is subjected to vertical differential ground movements at certain burial depths. In Joint #3, as shown in two linear trend lines in Figure 4-63, there is an influence of vertical ground displacement on joint rotation but to a lesser degree than in Joint #2. However, since the slopes of the two trend lines are almost identical, there appears to be only a miniscule effect on joint rotation of the two different burial depths (1.2 m versus 0.6 m) for these test conditions. It appears that burial depths had little effect on joint rotation for a given amount of differential ground movement for joints which fall a considerable distance on either side of the fault. In addition, based on work of Saiyar et al. (2010; 2015), there is a simplified trigonometric
relationship between ground displacement and joint rotation for cases where the differential
ground movements are concentrated within one pipe segment, as shown below.

\[ \beta = \sin^{-1} \left( \frac{\sigma_o}{L} \right) \]  
(Equation 4-1)

Where \( \beta \) is joint rotation (in degrees or radians), \( \sigma_o \) is ground displacement is millimetres, and \( L \) is the length of the pipe segment in millimetres. This theoretical relationship is also plotted in
Figures 4-62 and 4-63. For Joint #3, it appears that the joint rotation as a function of ground
displacement mimics the theoretical function closely regardless of burial depths as indicated by
the slopes of the linear trendlines. By contrast, in Joint #2 rotation behaviour only follows this
theoretical function in the case of shallow burial (0.6 m), but for deep burial depth the
relationship between joint rotation and vertical displacement deviates slightly from this
simplified theoretical trigonometric function.

The third analysis presented here explores the relationship between the axial movements across
the joints and the vertical ground displacements. Figures 4-64 and 4-65 present the axial joint
movements for both burial tests for Joint #2 and #3 as a function of vertical ground
displacement.
Figure 4-64 Joint #2 axial (horizontal) extension versus vertical ground displacement for tests at 1.2 and 0.6 m burial depths

\[
y = -6 \times 10^{-5}x^3 + 0.0029x^2 - 0.0002x - 0.0738 \\
R^2 = 0.9941
\]

Figure 4-65 Joint #3 axial horizontal extension versus vertical ground displacement for tests at 1.2 and 0.6 m burial depths

\[
y = -7 \times 10^{-5}x^3 + 0.0037x^2 - 0.0393x - 0.1134 \\
R^2 = 0.9123
\]
In Figures 4-64 and 4-65, it appears that joint axial (horizontal) extension exhibit an overall non-linear behaviour relative to vertical ground displacement. Polynomial functions of the third order were fitted for analysis between joint axial horizontal extension and vertical ground displacement (with R-squared values ranging between 0.912 and 0.997). The polynomial functions were not utilized in any subsequent computer modeling; instead, these functions were used to quantify the nonlinear behaviour of the joints’ axial horizontal extension and compression. For Joint #2 and Joint #3, there is a small extension/compression of the joint up to 2.1 to 4.0 mm of vertical displacements; after which an opposite horizontal axial extension (or compression) dominates in almost linear fashion.

From visual inspection, this transition occurred at different vertical ground displacements for different burial depths. For 1.2 m burial depth, it required approximately 2.3 mm of vertical displacement for this transitional horizontal axial movement to occur in both Joint #2 and #3. In contrast, for 0.6 m burial depth, an approximate vertical ground displacement of 2.6 mm was needed to trigger the transitional event in Joint #2. However, in Joint #3, this transitional event occurred at vertical ground displacement of approximately 3.2 mm. From this exercise, it could be suggested that burial depth had an effect on horizontal axial extension for the joint subjected (in this case, Joint #2) to vertical displacement. As indicated by the polynomial functions, as burial depth increased (i.e: from 0.6 m to 1.2 m), the rate of axial (horizontal) extension increased with a given amount of vertical displacement. This is indicated by the slopes of the fitted functions for the two burial depths of Joint #2 and Joint #3. In addition, the axial extension of Joint #1 is examined. Figure 4-66 displays the axial extension of Joint #1 in both burial conditions where the moving floor was subjected to up to 30 mm (not Joint #1 itself) of differential ground movement.
Figure 4-66 Joint #1 axial horizontal extension versus vertical ground displacement from 1.2 and 0.6 m burial tests

This figure shows that the extension of Joint #1 is miniscule; it is suspected that this horizontal extension represents the transitional event presented in Joint #2 and Joint #3 in Figure 4-64 and Figure 4-65. Given Joint #1 was under no DGM and situated at a considerable distance from the fault, this axial extension may be associated with gasket movement inside the joint. The almost complete recoveries of the axial horizontal extension at the conclusions of both tests seem to support this notion. This phenomenon is worth noting because it suggests that the joint can support a significant magnitude of vertical displacement (in this case, 10 to 15 mm) before the gasket can no longer tolerate and recover from axial horizontal extension.

The fourth analysis is the comparison between net axial extension of the pipe segment (subjected to differential ground movement, Test Pipe #3) versus trigonometric calculations. Based on Pythagoras' theorem, the axial extension of the pipe segment is the hypotenuse of the right-angle triangle minus the original pipe length (i.e., 1520 mm), Figure 4-67a. From this theoretical
calculation, the amount of pipe segment axial extension for 30 mm vertical displacement is approximately 0.30 mm. From the two burial tests, the axial extension of the pipe segment is the difference between the axial extensions of the two joints (i.e., Joint #2 and Joint #3) designated as net axial displacement. Figure 4-67b displays the theoretical calculation from Pythagoras' theorem and actual net axial displacements in both burial tests.
a. Theoretical length as Test pipe #3 rotates to accommodate differential ground movement $\delta$

$$T = \sqrt{\delta^2 + 1520^2}$$

Pipe length = 1520 mm

b. Net axial movement (theoretical versus actual)

Figure 4-67 Axial Extension of Test Pipe #3
Based on visual observation, it appears for 1.2 m burial test that the actual net axial displacement mimics the theoretical net axial displacement closely, especially at the moment of maximum vertical displacement (i.e, 30 mm), where actual net axial displacement is approximately 0.30 mm. In contrast, the actual net axial displacement for the 0.6 m burial test does not closely mimic the theoretical one. There might be additional physical mechanisms which interfered with the net axial extension motion during the test at 0.6 m burial depth, which had not been accounted for previously. The actual net axial displacement behaviour appears to be more erratic as compared to 1.2 m burial test, and net axial displacement is approximately 0.35 mm in contrast to 0.30 mm from theoretical calculations. In summary, both tests have demonstrated that pipe segment (i.e, Test Pipe #3) undergone axial extension during differential ground movement. There is a substantial net axial extension initially (until approximately 2.5 mm of vertical displacement) then the net axial extension starts to mimic the theoretical net axial extension prediction. This pipe axial extension conforms more closely to theoretical prediction for deeper burial (i.e, 1.2 m burial) and more erratic and higher magnitude for shallower burial (i.e, 0.6 m burial).

Lastly, a brief discussion of discrepancies between joint rotation measurements by LPs and inclinometers is presented. As observed in Sections 4.7 and 4.15, there are discrepancies between the joints’ rotation measurements obtained from LPs and inclinometers. Additional physical mechanisms are suspected for these deviations. One possibility is the bending of the pipe segments during DGM. All inclinometers are situated inside the bell portion of the joints with an offset of approximately 50 mm. The bending of the pipe segments will change the slope of the spigot end of each pipe relative to its bell, and this means that the rotations obtained by inclinometers do not represent the rotation along the whole of the pipe. The strain data from both tests confirm that Test Pipes #2 and #3 underwent significant strain during DGM, and would
therefore have experienced curvature. Figure 4-68 is a visual idealization of this pipe bending mechanism.

Figure 4-68 Exaggerated idealization of the pipe bending mechanism

Another physical mechanism which could contribute to the discrepancies between rotation values estimated using LPs and inclinometers is associated with the shear forces that develop across the joints during DGM. Those shear forces could cause some gasket compression, and this produces small amounts of rotation. Figure 4-69 is the idealization of this shearing mechanism.
Since the instrumentation scheme did not feature any vertical LP measuring shear deformation (i.e. gasket compressions) across each joint, it is not clear to what extent that gasket compression under shear contributed to the measurement discrepancies. Therefore, measurement of vertical joint displacement is suggested for subsequent investigations.

4.19 Summary and Conclusions

In summary, the pipe assembly was buried at depths of both 1.2 and 0.6 m and tested under the differential ground movements associated with a normal ground fault. The test chamber performed as designed to simulate a normal fault involving a total of 30 mm of vertical displacement. Olivine sand was compacted using gas-powered vibratory plate compactor, producing dry density of between 1530 to 1750 g/cm³ with an average dry density between 1677
and 1693 g/cm³ and standard deviation between 38 and 46 g/cm³ for both burial tests. For hand-tamped lifts at pipe level, the dry density averaged between 1570 and 1580 g/cm³ for both tests with a standard deviation between 24 g/cm³ and 48 g/cm³. The moisture content of the olivine sand averaged between 0.33 and 0.63% for the two tests, so for the purposes of soil placement, compaction, and testing, the olivine sand is deemed to be essentially dry.

Joint movements were monitored using linear potentiometers placed within the pipes, and crossing the joints along the crown and pipe haunches. These indicated that for both tests, most of the differential ground motion was accommodated by one pipe segment (Test Pipe #3 which straddled the shear band caused by the normal ground fault). The joints on either end of that pipe (Joints #2 and #3) had rotations that were almost equal and opposite (rotations of -1.15 degrees and 0.99 degrees respectively for burial depth of 1.2m, and -1.18 degrees and 1.07 degrees for burial depth of 0.6m). The rotations of each pipe segment were also monitored using slope indicators attached near to each bell. Differences in these measurements between each pipe gave almost the same values of joint rotation as those obtained from the LP measurements. However, joint rotations obtained from the slope indicator measurements were somewhat lower than those obtained from LPs, likely because the pipe segments did experience some longitudinal bending (they are not truly ‘rigid’ pipes).

Joint contraction and extension was also monitored using the linear potentiometers. For the test at 1.2m burial depth, Joint #2 experienced compression of 0.85mm, while Joint #3 experienced an extension of 1.17mm. For the test at 0.6m burial depth, Joint #2 experienced compression of 0.12mm, while Joint #3 experienced extension of 0.51mm. For both burial cases, Joint #1 Joint #1 had a maximum axial extension of less than 0.025mm, and this almost near completely recovered by the end of each test. Use of Pythagoras’ theorem suggests that a net extension of
close to 0.3mm would be expected in each case. Longitudinal strain measurements along the crown and invert of the test pipes were reported from both tests. Those from the 1.2 m burial test were significantly higher than those from the 0.6 m burial test. Both Test Pipe #2 and Test Pipe #3 experienced significant strain levels – likely because the normal fault was positioned below Joint #2 where these two pipes were joined. Each of the strain measurements at the crown was almost equal and opposite to the invert value at that axial position in that test pipe, an indication that strain was associated with longitudinal bending and that little total axial force developed across the circular cross section of the test pipes.

Post-test inspection of both burial tests indicated that the friction treatment deployed around the walls of the test chamber performed as intended. The top surface of the friction treatment around the moveable portion of the test chamber had displaced approximately between 3.5 to 4.0 cm around all three walls (i.e: Eastern, Western, and Southern). In contrast, around the stationary portion of the test chamber, the top surface displacement was between 0.6 and 0.8 cm – the amount of vertical soil compression that must have occurred during soil placement and compaction.

External stringpot measurements indicated that only minuscule horizontal movements of the extreme ends of the pipe assembly occurred during both tests. Pre-test and post-test elevation readings of the pipe assembly for both tests were also recorded. From both tests, Test Pipe #4 (and its joints) was vertically displaced by an amount almost equal to the imposed ground movements. In the 1.2 m burial test, Test Pipe #4, the vertical displacement ranged between 19 and 38 mm. For the 0.6 m burial test, the vertical displacement ranged between 29 and 41 mm for this pipe (and associated joints). In contrast, the vertical displacement for Test Pipes #1 and
#2 only ranged between 1 to 12 mm for the two pipes (and associated joints) for both the 1.2 m and 0.6 m burial tests.

The relationship between joint rotation and vertical ground movement was analyzed by plotting the rotations of Joints #2 and #3 versus vertical ground displacement of the moving floor frame. These indicate that depth of burial had little influence on joint rotation for a given amount of ground displacement, and indicated that the joint rotations were approximately linear functions of vertical ground displacement.

The relationship between axial joint extension or contraction and the vertical ground displacement was also analyzed by plotting those axial joint movements for all three joints (of both burial tests) versus vertical ground displacement. These plots revealed that joint extension or contraction were non-linear functions of the vertical ground displacement. Those functions were different for the experiments conducted at the two different burial depths, with the test at greater burial depth featuring higher movements (likely because of the closer proximity of the pipe to the ground fault imposed on the floor below). In addition, both figures indicate that there is a transitional event where the joint initially undergoes a small degree of axial extension/compression, then an opposite and more significant axial (horizontal) movement. This transition event occurred at approximately 2.3 mm of vertical displacement for the test at 1.2 m burial depth. For the 0.6 m burial test, the transition occurred at vertical ground displacement of 2.6 mm for Joint #2m and 3.2 mm for Joint #3. Therefore, it appears that burial depth has some influence on the initiation of this change in behaviour. For joint situated away from the fault (i.e: Joint #1) there appears to be a recovery of the axial movement after the full 30 mm of vertical fault displacement.
A geometrical estimate of the net extension of the pipeline provided net axial extension of 0.3mm, similar to that seen in the experiment. However, development of the net extensions during the tests was somewhat erratic.

Discrepancies were observed in the joint rotations obtained from the inclinometers and linear potentiometers (LPs), likely as a result of pipe bending between the joints, a notion that was supported by bending strains observed during the tests. It may also be that vertical deformations imposed on gaskets within the joints caused further discrepancies, though no instruments were used to measure shear deformations and further testing is needed to quantify how gasket compression under shear contributed to joint rotations.
4.20 References


Chapter 5 Summary and Conclusions

5.1 Summary

This chapter summarizes conclusions drawn from the works of this thesis, these include: joint monitoring system, full scale articulation test, test chamber simulation of normal ground fault, and both shallow and deep burial tests.

A joint instrumentation scheme composed of linear potentiometers, inclinometers and strain gauges was designed to monitor a pipe assembly of four pipes. The four pipes had never been utilized in the field. Three pipe joints were monitored by LPs and inclinometers for joint rotations and joint axial translation. All four pipes were instrumented with strain gauges at both crown and invert positions.

To impose a normal faulting differential ground movement to the pipe assembly, a test chamber was designed and constructed. The test chamber is 1.8 m wide, 7.3 m in length and 1.8 m deep. A moveable floor was used to simulate differential ground movement of 30 mm; four manually operated screw jacks provided the support and the mechanism for this ground displacement. The test chamber was subjected to a trial test before full-scale tests involving the pipe assembly were conducted.

Shortly after the test chamber trial, the pipe assembly was subjected to 30 mm of differential ground movement twice in separate tests. The olivine sand was compacted using a gas operated vibratory plate to achieve a relatively high dry density between 1650 to 1720 g/cm³. Data collected from these two tests were analyzed and compared for qualitative and quantitative interpretation of joint behaviours in olivine sand under imposed normal differential ground movement.
5.2   Pipe Sample, Joint Monitoring System, and Full-Scale Articulation Test

After the full-scale articulation test, it appears that the LPs and inclinometers performed within expectations. The joint rotation difference measured between LPs and inclinometers was between 0.009 and 0.124 degree. At maximum vertical displacement from this full-scale articulation, the percentage differences between inclinometers and LPs was between 0.16 and 8.5%; a difference of less than 10%.

From total station readings, there appeared to re-alignment of all joints post-vertical displacement of 25 mm. Joint #2 as expected underwent the highest amount of lateral movement as it was the only point of lifting during the full-scale articulation test. Joint #1 underwent the least amount of lateral movement, however, total station reading also measured lateral movement in Joint #3 comparable to those experienced by Joint #2. The lack of lateral soil restraint was the probable cause for the lateral movement of the joints during the articulation test.

Most of the strain gauges performed effectively and reported strain data during the full-scale test, and only three of the twenty-four gauges malfunctioned. These three gauges either reported offscale readings or accumulation of strain. Therefore, these strain gauges were utilized for the two subsequent burial tests to measure strain along the pipe segments.

5.3   Test Chamber for Simulating Normal Ground Faults

Overall, the test chamber performed as intended when tested on May 24, 2015. The chamber is 1.8 m wide, 7.3 m long, and 1.8 m deep. The test chamber has a moveable floor along the Southern half and a stationary floor along the Northern half. Four screw jacks supported and controlled the vertical displacement of the moveable floor. Friction treatment was applied on all four walls of the test chamber to reduce angle of friction to approximately 4 degrees. The width
of this test chamber of 1.8 m was chosen to prevent any pipe uplift influencing soil movements along the test chamber’s walls.

The soil only test conducted on May 24, 2015 was also used to examine the effectiveness of the sealing system to prevent sand leakage for subsequent burial tests. This chamber demonstrated to be capable of producing differential ground movements totaling 122 mm of vertical ground displacement. Post-test inspection confirmed that the sealing system had performed within the design intent, since no olivine sand leakage had occurred during the soil-only test.

In addition, sand was compacted with vibrating plate compactor and this compaction method produced olivine sand dry densities between 1650 and 1720 g/cm³. The average olivine sand dry density from this soil only test was 1677 g/cm³.

5.4 Full Scale Tests at 1.2 m and 0.6 m Burial Depths

Two full-scale burial tests were conducted by using the test chamber to simulate normal ground faulting, which created a total differential ground movement of 30 mm for both tests. Olivine sand was used for both tests. Compacted was undertaken using a gas-powered vibratory plate compactor, which produced a dry density between 1530 to 1750 g/cm³ with an average of approximately 1677 g/cm³ and a standard deviation of 46 g/cm³ for both burial tests. The olivine sand was hand-tamped using steel plates for the two lifts at pipe level and immediately above pipe level. The hand-tamping produced a dry density averaging between 1570 and 1580 g/cm³ for both tests with a standard deviation between 24 g/cm³ for the hand-tamped lifts for test at 0.6 m burial depth and 48 g/cm³ for the test at 1.2 m burial depth. The olivine sand was deemed essentially dry for the purposes of soil placement, compaction, and testing due to moisture content averaging between 0.33 and 0.63% for both tests.
Linear potentiometers placed within the pipes monitored three joints of the pipe assembly. Inclinometers were also placed near each bell of the pipe segments to monitor rotations in real-time during testing. As indicated by inclinometers and linear potentiometers, the majority of the differential ground movement was absorbed by only one pipe segment, Test Pipe #3, because it was located in the shear band zone caused by the normal ground faulting. The two joints associated with this test pipe, Joint #2 and #3, had almost equal and opposite rotation, which confirmed this notion. There was a small difference between joint rotations as obtained by inclinometers and linear potentiometers. This could be due to the pipe segments (particularly, Test Pipe #3) experienced longitudinal bending which suggests the extent to which these pipes are not fully rigid.

The axial contraction/extension was monitored by the linear potentiometers crossing all three joints. Joints #2 and #3 experienced the highest degree of axial contraction/extensions. Joint #1 had a maximum extension of less than 0.025 mm and recovered this extension completely by the end of both tests. Joint #2 experienced a contraction of 0.85 mm for the test at 1.2 m burial depth and 0.12 mm for the test at 0.6 m burial depth. In comparison, Joint #3 experienced an extension of 1.17 mm for the test at 1.2 m burial depth and a extension of 0.51 mm for the test at 0.6 m burial depth.

Strain gauges on all four test pipes located along the crown and invert positions reported longitudinal strain. As expected, the strains reported from the test at 1.2 m burial depth were significantly higher than those measured from the 0.6 m burial depth test. Measurements at the crown were almost equal and opposite to those at inverts at corresponding axial positions. This is an indication that the pipe segments were largely influenced by longitudinal bending, rather than axial thrust.
Post-test examinations from both tests indicated that the friction treatments on all four walls of the test chamber performed as intended. The friction treatment on the moveable portion (i.e: Eastern, Western, and Southern) of the test chamber had displaced vertically by approximately 3.5 to 4.0 cm. In comparison, the stationary side had displaced by approximately 0.6 and 0.8 cm for the test at 1.2 m burial depth, and approximately 1.0 to 2.5 cm for the test at 0.6 m burial depth (this could be due to a brief separation of friction treatment layers during backfilling), however, the small vertical displacement of the friction treatment for the test at 1.2 m burial depth is an indication that soil compression had occurred during its placement and compaction. The stringpot which was attached externally to the pipe assembly indicated that little horizontal axial motion occurred for both burial tests. In addition, elevation readings taken pre-test and post-test were compared. Test Pipe #4 experienced the highest degree of vertical displacement during both tests. The pipe segment and its associated joints had experienced between 19 mm and 38 mm of vertical displacement for the test at 1.2 m burial depth, and between 29 and 41 mm for the test at 0.6 burial depth, indications that this pipe displaced by approximately the same amount as the vertical differential ground movement (i.e: 30 mm).

There is very little influence of burial depth on vertical displacement and joint rotation seen in the test data presented and analyzed in Chapter 4.

The axial joint extension/contraction versus vertical ground displacement was analysed. All three joints for both burial tests were studied. It is observed that joint extension/contraction operate in a non-linear fashion in relation to vertical ground displacement. Higher axial movements resulted in the pipe at greater burial depths. This is a reasonable result as the deeper the burial depth, the closer the proximity the pipe assembly (and its joints) to the fault. From the data, there appears to be a transitional event where the joint initially experiences a small amount of axial
compression/extension before succumbing to a dominant axial movement in the opposite
direction. In the test at 1.2 m burial depth, this transition event occurred at 2.3 mm of vertical
displacement compared to approximately 2.6 mm (for Joint #2) and 3.2 mm (for Joint #3) for the
test at 0.6 m burial depth. It appears that burial depth has a small effect on the onset of this joint
axial transitional event. Net axial extension versus vertical displacement was analyzed, and for
the test at 1.2 m burial depth, the net extension followed the theoretical net axial displacement
(as calculated by Pythagoras’ theorem) closely but for the test at 0.6 m burial depth, this
behaviour was somewhat erratic.

As seen and discussed in Chapter 4, discrepancies between joint rotations estimated from linear
potentiometer and inclinometer readings could be attributed to pipe bending, and significant
longitudinal strains were seen during both tests. In addition, vertical deformation of the gaskets
due to shearing could also be attributed to these discrepancies.

5.5 References

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Appendix A Strain gauging procedure on vitrified clay pipes

This section provides details concerning installation procedure of strain gauges on vitrified clay pipes.
This appendix details the procedure undertaken to adhere strain gauges onto vitrified clay pipes. There are two stages for strain gauging, surface preparation and application of epoxy onto strain gauges. Both stages follow instructions by Micro-Measurements Group. For surface preparation, the author followed Instruction Bulletin B-129-8. For application of AE-10 epoxy, Instruction Bulletin B-137 was followed.

The first stage is surface preparation of the clay pipe, refer to Table II in Bulletin B-129-8 in terms of Specimen Material. Ceramic was chosen as the Specimen Material as recommended by Vishay's representative. The following is the procedure for surface preparation (for additional details refer to Bulletin B-129-8):

1. Lay out the location of the strain gauge with two perpendicular lines forming a crosshair. A thin-tipped marker is highly recommended.

2. Use an electric grinder to polish the surface. The area of polishing should be more than 3 to 4 times the size of strain gauge. For Kyowa strain gauges (with a gauge factor of 2.12 +/- 1.0%). The strain gauges used on the vitrified clay pipes were 5 mm in length with a gauge resistance of 119.8 +/- 0.2 ohms (Ω). The polished area should not be bigger than thumb's area. Gently polish area on low speed. This removes any foreign object on the pipe. The polished area should then feel and appear smooth, grind the area with the electrical grinder 4 to 5 times..

3. Gently abrade the polished area with silicon-carbide paper. The 320-grit paper is used for clay pipe. This serves as a final stage of surface abrading, and ensures that no foreign particle is on the surface.

4. Wipe polished area with Isopropyl alcohol using clean and disposable paper wipes. Wipe the area in one direction ONLY, DO NOT wipe the area back and forth, since this might
re-contaminate the area. Avoid touching the area with fingers, since this would reintroduce grease onto the area. DO NOT reuse paper wipes. If there is no discolouration on the paper wipes, this process is finished.

5. Redraw strain gauge layout if necessary, use the isopropyl alcohol to decontaminate ruler before placing it onto the pipe surface.

6. Apply Conditioner A supplied by Micro-Measurements Group on the surface. Use clean, disposable paper wipes for application. Wipe area in one direction ONLY. DO NOT wipe the area back and forth; use a clean wipe for each application. Continue this process until no discolouration is observed on the paper wipe. DO NOT touch area with fingers.

7. Apply the M-Prep Neutralizer 5A supplied by Micro-Measurements Group on the conditioned surface. Use clean, disposable paper wipes for application. Wipe area in one direction ONLY. DO NOT wipe the area back and forth; use a clean wipe for each application. Continue this process until no discolouration is observed on paper wipe. DO NOT touch area with fingers.

The second stage is application of AE-10 epoxy. Please refer to Instruction Bulletin B-137 from Micro-Measurements Group for detailed instructions on strain gauge preparation. The installation procedure for strain gauges on vitrified clay pipes is the following:

1. Prepare a small amount of AE-10 epoxy to the correct portion of resin and curing agent. The following formula is used to determine amount of curing agent by mass:

   \[ Curing \ agent \ (g) = \frac{Resin \ (g)}{10} \times 1.5 \]

2. Apply a small amount of AE-10 onto prepared location for strain gauging. Roll a small piece of MJG-2 Mylar tape supplied by Micro-Measurements Group. This is
done to create a thin film of epoxy on the strain gauge location. The following figure presents this preparation step. Let the epoxy stand for 24 hours.

![Figure A-1 Thin film of AE-10 epoxy](image1)

3. Remove tape and expose thin film of epoxy as depicted below.

![Figure A-2 Exposure of AE-10 epoxy](image2)
4. Gently abrade thin film of epoxy with 320-grit paper. Be careful not to remove the thin film of epoxy. This step is only to roughen the epoxy-ed surface as depicted below.

![Figure A-3 Roughened epoxied surface](image)

5. Place prepared strain gauge onto the pipe. Refer to Instruction Bulletin B-137 regarding proper strain gauge preparation. Apply a small amount of AE-10 epoxy. Roll the tape gently onto the pipe, ensuring that the epoxy completely engulfs the strain gauge.
6. Place rubber membrane on top of the strain gauge. Add a weight to apply firm pressure on the strain gauge to ensure proper bonding. Maintain a firm pressure for approximately 8 hours. The author used a ratchet strap to maintain pressure as depicted below.
Appendix B Articulation test summary with Thorlab rails

This appendix summarizes a smaller scale articulate test to verify the functionality of the Thorlab rails for securing the linear potentiometers inside the pipe joint.
An articulation test was conducted on October 21 2014. An aluminum rail system from Thorlab was used to mount the linear potentiometers. Three linear potentiometers (LPs) were used to monitor each joint; the LPs were spaced 120 degrees apart. In the following figure depicts the LP layout.

![Figure B-1 Linear potentiometer layout](image)

One LP was placed at the crown position at the spigot portion of pipe, and two other LPs were at the haunch positions. The dimensions of LP layout is provided below, dimensions are in millimeters.
A wooden contact plate was fixed to the bell portion of pipe as depicted below.

Figure B-2 Linear potentiometer layout dimensions
A scissor car jack was placed underneath the joint connection to elevate joint. The joint connection was elevated to a height of approximately 25.4 mm. A wire stringpot was placed directly above the middle of the joint to record vertical displacement. Both ends of the pipe assembly were secured onto cinder blocks using ratchet straps, which ensured no axial movement during lifting. Each end of the pipe assembly was supported approximately 147.32 cm (58 inches) relative to centre of the joint. The test’s external configuration is displayed below.
Inclinometers were also placed inside the bell portion of these two pipes. These inclinometers were to monitor degree of inclination during the articulation test. A total of two trials were conducted. Each trial had four lifting steps; the joint was lifted to pre-determined height of approximately 25.4 mm and lowered back to original position. The joint was held in elevated position for two minutes to stabilize readings.

Unfiltered inclinometer data from the two trials are provided below. Note that one inclinometer (J2_Inclinometer), in Test Pipe #2, had a loose mount which caused erroneous readings; its data had been omitted.
Based on previous experience with the inclinometers, the author is confident with inclinometer precision and accuracy. From the raw data, the joint had a rotation relative to vertical plane approximately between 2.54 and 2.66° degrees.
Unfiltered data from the linear potentiometers during the two trials are presented below.

Figure B-7 Linear potentiometer readings from articulation test #1

Figure B-8 Linear potentiometer readings from articulation test #2
Appendix C Movable floor and components design calculations

This appendix presents the design calculations the moving floor frame and its associated components, these include: bolts (for frame and screw jack connections), the steel joist, longitudinal beams, and transverse beams. All calculations are conducted in collaboration with Mr. Pengpeng Ni.
C-1 Steel bolt design calculations

**Factor of Safety Calculations: All calculations are in kN, kPa, m**

Volume friction angle:
\[ \phi_{\text{sand}} := 34 \]

Lateral earth pressure coefficient:
\[ k_o := 1 - \sin\left( \phi_{\text{sand}} \cdot \frac{\pi}{180} \right) = 0.441 \]

Unit weight:
\[ \gamma_{\text{sand}} := 15.8 \]

Height of soil:
\[ H_{\text{sand}} := 1.83 \]

Lateral force from soil per meter:
\[ P_{\text{sand}} := 0.9 k_o \gamma_{\text{sand}} H_{\text{sand}}^2 = 11.66 \]

Lateral force at the floor level per meter:
\[ P_{\text{bottom}} = 2P_{\text{sand}} = 23.324 \]

Length of the pit:
\[ \text{Length}_{\text{feet}} := 1.8288 \]

Lateral force at the floor level:
\[ F_{\text{bottom}} = P_{\text{bottom}} \cdot \text{Length}_{\text{feet}} = 42.655 \]

Force on each bolt (bottom two):
\[ F_{\text{bolt}} := 0.9 F_{\text{bottom}} = 21.328 \]

Diameter of bolt (1/2 nominal diameter):
\[ d := 0.0127 \]

Area of bolt:
\[ A := \frac{\pi d^2}{4} = 1.267 \times 10^{-4} \]

Shear stress:
\[ \tau \leq \frac{F_{\text{bolt}}}{A} = 1.684 \times 10^6 \]

Nominal strength in shear:
\[ \tau_{\text{nominal}} = 496422.525 \]

In general, the tensile strength of a grade 5 fastener is 120,000 psi (https://www.boltdepot.com/fastener-information/materials-and-grades/bolt-grade-chart.aspx). The shear strength is usually about 50-60% of the tensile strength. We took 60% of the tensile strength in our calculation.

Safety factor:
\[ SF := \frac{\tau_{\text{nominal}}}{\tau} = 2.949 \]
Steel Joist Calculations

Unit: length in m, force in kN, stress in kPa, moment in kN.m

Soil load:
Height := 1.8
Width := 1.8288
Length := 0.3048
FOS := 2
Unitweight := 15.8

\[ q := \frac{\text{FOS \times Unitweight \times Height \times Width \times Length}}{\text{Width}} = 17.337 \]

BeamWidth := 0.0064

\[ \text{S}_{\text{timber}} := \frac{\text{Width} - 0.8}{2} + \frac{\text{BeamWidth}}{2} = 0.518 \]

\[ L_{\text{timber}} := 0.8 - \text{BeamWidth} = 0.794 \]

\[ W_S := \text{FOS \times Unitweight \times Height \times S}_{\text{timber}} \times \text{Length} = 8.974 \]

\[ W_L := \text{FOS \times Unitweight \times Height \times L}_{\text{timber}} \times \text{Length} = 13.759 \]

Maximum shear and moment for the timber:

\[ R_a := q \left( \text{S}_{\text{timber}} + \frac{L_{\text{timber}}}{2} \right) = 15.853 \]

\[ V_A := \max \left( q \cdot S_{\text{timber}}, q \cdot \frac{L_{\text{timber}}}{2} \right) = 8.076 \]

\[ M_A := \frac{q \cdot S_{\text{timber}}^2}{2} = -2.322 \]

\[ M_D := \frac{q \cdot L_{\text{timber}}^2}{8} + M_A = -0.958 \]

Design target:

\[ \sigma_y := 345000 \]

\[ Z := \frac{\max \left| M_A \right|, \max \left| M_D \right|}{\sigma_y} = 6.732 \times 10^{-6} \]

Select beam section, HSS 51x51x3.2 from Handbook of Steel Construction, Eighth Edition, Hollow Structural Sections, CSA G40.20 Square, Page 6-103:

\[ y := 0.051 \]

\[ I := 0.214 \times 10^{-6} \]

\[ \sigma_{fy} := \frac{\max \left| M_A \right|, \max \left| M_D \right| \cdot y}{2I} = 2.767 \times 10^5 \]
Dead load of this joist (kN/m):

\[ w := 0.045 \]
\[ F_{\text{outframe}} := w \times \text{Length} \times \text{FOS} = 0.027 \]
\[ M_{\text{A,outframe}} := F_{\text{outframe}} \times S_{\text{timber}} = 0.014 \]
\[ M_{\text{A}} := |M_{\text{A}}| + M_{\text{A,outframe}} = 2.337 \]
\[ \sigma := \frac{M_{\text{A}} y}{2I} = 2.784 \times 10^5 \]
C-3 Longitudinal beam design calculations

**Unit:** length in m, force in kN, stress in kPa, moment in kN.m

**Soil:**
Height := 1.8
Width := 1.8288
Length := 0.3048
FOS := 1.8
Unitweight := 15.8
\[ q := \frac{\text{FOS} \times \text{Unitweight} \times \text{Height} \times \text{Width} \times \text{Length}}{\text{Width}} = 15.603 \]

**Longitudinal beam section:** HSS 64x64x64

**Beam Width:** := 0.0064

\[ S_{\text{timber}} := \frac{\text{Width} - 0.8}{2} \]

\[ L_{\text{timber}} = 0.8 - \text{Beam Width} = 0.794 \]

**Maximum shear and moment for the timber:**

\[ R_a = q \left( S_{\text{timber}} + \frac{L_{\text{timber}}}{2} \right) = 14.268 \]

\[ V_A := \max \left( q \times S_{\text{timber}}, q \times \frac{L_{\text{timber}}}{2} \right) = 8.076 \]

\[ M_A := \frac{q \times S_{\text{timber}}^2}{2} = -2.09 \]

\[ M_D := \frac{q \times L_{\text{timber}}^2}{8} + M_A = -0.862 \]

The reaction force \( R_a \) from the timber will act on the longitudinal beam, as \( F \) in the following.

**Joist (HSS 51x51x3.2):**

**Dead load of this joist (kN/m):**

\[ y := 0.051 \]

\[ w := 0.045 \]

\[ F_{\text{joint}} := w \times \text{Width} \times \text{FOS} = 0.148 \]

\[ F_{\text{outframe}} := 2w \times (\text{Length} - y) \times \text{FOS} = 0.041 \]

**Concentrated load:**

\[ F_{\text{concent}} := R_a + 0.5 \times (F_{\text{joint}} + F_{\text{outframe}}) = 14.362 \]

**Longitudinal beam analysis:**

13 concentrated loads are distributed along the longitudinal beam, which has two supports. We can use **superposition** to get the total moment diagram from each concentrated load.

Three load case:
Case I, concentrated load is acting at the left side of the support.

Case II, concentrated load is acting between two supports.

Case III, concentrated load is acting at the right side of the support.

\[
\begin{array}{c|c|c}
\text{Case I} & \text{Case II} & \text{Case III} \\
\hline
F & F & F \\
R_1 & R_2 & \\
\end{array}
\]

In this figure, the concentrated load is denoted as \( F \), and the reaction forces are represented by \( R_1 \) and \( R_2 \). The total length of the longitudinal beam is \( L \), the distance from the support to the beam end is \( S \). The left side is the original coordinate, \( 0 \). \( F \) is acting at a distance \( x \) from the origin, \( y \) is the distance along the beam.

For three loading cases, the calculation of reaction forces is the same:

\[
R_1 := -F \frac{L - x - S}{L - 2S} \quad R_2 := -F \frac{x - S}{L - 2S}
\]

Calculation of bending moment distribution along the beam:

The concerned sectional moment at a distance of \( y \) from left end can be calculated as follows:

Case I, concentrated load is acting at the left side of the support.

\[
\begin{align*}
y < x & \quad M(y) = 0 \\
x < y < S & \quad M(y) = F(y - x) \\
S < y < L - S & \quad M(y) = F(y - x) + R_1(y - S) \\
L - S < y & \quad M(y) = 0
\end{align*}
\]

Case II, concentrated load is acting between two supports.

\[
\begin{align*}
y < S & \quad M(y) = 0 \\
x < y < x & \quad M(y) = R_1(y - S) \\
S < y < L - S & \quad M(y) = F(y - x) + R_1(y - S) \\
L - S < y & \quad M(y) = 0
\end{align*}
\]

Case III, concentrated load is acting at the right side of the support.

\[
\begin{align*}
y < S & \quad M(y) = 0 \\
S < y < L - S & \quad M(y) = R_1(y - S) \\
L - S < y < x & \quad M(y) = R_1(y - S) + R_2[y - (L - S)] \\
L - S < y & \quad M(y) = 0
\end{align*}
\]

From Matlab, the maximum bending moment is 16.3 kN.m

\[
M_{\text{max}} = 14.21
\]

For the HSS 64x64x64 beam, it has:

\[
\gamma_{\text{neutral}} := 0.032 \quad \frac{M_{\text{max}}}{345 \cdot 1000} = 4.119 \times 10^{-5}
\]

\[
I := 0.663 \cdot 10^{-6}
\]

We are going to add one beam beside the original beam, so the neutral axis distance will not change, but the moment of inertia will be doubled.
\[ \sigma := \frac{M_{\text{max}}}{2I} y_{\text{Neutral}} = 3.429 \times 10^5 \]

The maximum stress is approximately 342.9 MPa, which is close to the design target of 350 MPa, but we have a factor of safety of 1.8. Besides, the assumption is the supports from the transverse beam are concentrated at two locations. In reality, we have two transverse beams at one location, which supplies a spread area to withstand 13 pairs of concentrated loads from timber/soil volume above. The spread area will help decreasing the moment arm of each concentrated load, and the total maximum bending moment would be decreased accordingly. Therefore, our design assumption is still conservative.
C-4 Transverse beam design calculations

All Calculations are in kN, m and kPa

\[
\begin{align*}
\text{width} & := 1.83 & \text{Soil weight} & := 15.8 \\
\text{height} & := 1.8 & \text{FOS} & := 2 \\
\text{length} & := 3.65 \\
\text{soil load} & := \text{width} \times \text{length} \times \text{height} \times \text{Soil weight} = 189.965 \\
\text{factored soil load} & := \text{soil load} \times \text{FOS} = 379.93 \\
\text{point load} & := \frac{\text{factored soil load}}{2} = 189.965 \\
\text{Two sets of transverse beam} \\
\text{Based on Dimension from drawings} \\
\text{a} & := 0.1 & \text{length between jacks} & := 0.736 \\
\text{Mmax} & := \text{point load} \times a = 18.996 & \text{Ra} & := \text{point load} = 189.965 \\
\end{align*}
\]

Max moments and reaction force from simple beam analysis

89 by 89 by 9.5 HSS (Existing frame)

Moment of inertia is 2 times, 2 transverse beams put side by side, moment arm remains the same

\[
\sigma := \frac{\text{Mmax} \times 0.0445}{2 \left(2.65 \times 10^{-6}\right)} = 1.595 \times 10^5 \\
\text{Transverse beam is under 250 MPa design consideration}
\]
C-5 Moving floor drawing (frame dimensions are in inches)

- Floor Deck, HSS 51x51x3.2
- Add second HSS 64x64 inside existing
- Bolted connection to existing longitudinal beam using flat head bolts
Appendix D CAD drawings and design calculations of test chamber

Two test chamber concrete block configurations with associated factors of safety are presented in this section. All calculations and designs are performed with collaboration with Mr. Pengpeng Ni.
D-1 Factor of safety calculations for moveable and stationary portions of test chamber.

**Factor of Safety Calculations: All calculations are in kN, kPa, m**

- **Volume friction angle:**
  \[ \phi_{\text{sand}} = 34 \]

- **Lateral earth pressure coefficient:**
  \[ k_o = 1 - \sin\left(\phi_{\text{sand}} \frac{\pi}{180}\right) = 0.441 \]

- **Unit weight:**
  \[ \gamma_{\text{sand}} = 15.8 \quad \gamma_{\text{gran}} = 23.5 \quad \gamma_{\text{con}} = 24 \]

- **Friction coefficient:**
  \[ \mu = 0.3 \]

- **Height of soil:**
  \[ H_{\text{sand}} = 1.83 \quad H_{\text{gran}} = 1.22 \]

- **No. 1 concrete block (3 foot) dimension:**
  \[ \text{Width}_{\text{con}} = 0.914 \quad H_{\text{con}} = 0.609 \quad \text{Length}_{\text{con}} = 0.914 \]

- **Weight of No. 1 concrete block:**
  \[ \text{Weight}_{\text{block}} = \text{Width}_{\text{con}} \cdot H_{\text{con}} \cdot \text{Length}_{\text{con}} \cdot \gamma_{\text{con}} = 12.21 \]

- **Lateral force from soil:**
  \[ P_{\text{sand}} = 0.5k_o \cdot \gamma_{\text{sand}} \cdot H_{\text{sand}}^2 = 11.662 \]
  \[ P_{\text{total}} = 0.5k_o \cdot \gamma_{\text{sand}} \cdot H_{\text{sand}}^2 + k_o \cdot \gamma_{\text{sand}} \cdot H_{\text{sand}} \cdot H_{\text{gran}} + 0.5k_o \cdot \gamma_{\text{gran}} \cdot H_{\text{gran}}^2 = 34.921 \]

- **Length of the pit:**
  \[ \text{Length}_{1\text{feet}} = 0.9144 \quad \text{Length}_{2\text{feet}} = 2.7432 \quad \text{Length}_{12\text{feet}} = 3.6576 \]

**Stationary Portion: Against Sliding**

- **At the base:**
  \[ n_{\text{first}} = 16 \quad n_{\text{second}} = 15 \quad n_{\text{third}} = 10 \]
  \[ F_{\text{soil}} = P_{\text{total}} \cdot \text{Length}_{2\text{feet}} + P_{\text{sand}} \cdot \text{Length}_{1\text{feet}} = 106.459 \]
  \[ F_{\text{concrete}} = \mu \cdot \text{Weight}_{\text{block}} \cdot (n_{\text{first}} + n_{\text{second}} + n_{\text{third}}) = 150.185 \]
  \[ \text{FOS} = \frac{F_{\text{concrete}}}{F_{\text{soil}}} = 1.411 \]

- **At the interface between granular A and Olivine sand:**
  \[ n_{\text{first}} = 9 \quad n_{\text{second}} = 9 \quad n_{\text{third}} = 6 \]
  \[ F_{\text{soil}} = P_{\text{sand}} \cdot \text{Length}_{12\text{feet}} = 42.655 \]
  \[ F_{\text{concrete}} = \mu \cdot \text{Weight}_{\text{block}} \cdot (n_{\text{first}} + n_{\text{second}} + n_{\text{third}}) = 87.913 \]
$F_{OS} = \frac{F_{concrete}}{F_{soil}} = 2.061$

**Movable Portion: Against Sliding**

$n_{first} = 9 \quad n_{second} = 4 \quad n_{third} = 4$

$F_{soil} = P_{sand} \times \text{Length}_{12\text{feet}} = 42.655$

$F_{msoil} = \mu \times \text{Weight of block} \times (n_{first} + n_{second} + n_{third}) = 62.272$

$F_{OS} = \frac{F_{concrete}}{F_{soil}} = 1.46$

**Stationary Portion: Against Rotation**

At the base:

$n_{first} = 16 \quad n_{second} = 15 \quad n_{third} = 10$

$A_{m_{first}} = \frac{1}{2} \quad A_{m_{second}} = \frac{1}{2} \quad A_{m_{third}} = \frac{1}{2}$

$M_{sand} := P_{sand} \times \text{Length}_{12\text{feet}} \times \left(\frac{1}{3}H_{sand} + H_{gran}\right) = 78.059$

$M_{sandgran} := (k_{o} \times \gamma_{gran}H_{sand}H_{gran}) \times \text{Length}_{9\text{feet}} \times \left(0.5H_{gran}\right) = 26.02$

$M_{gran} := (0.5k_{o} \times \gamma_{gran}H_{gran}^{2}) \times \text{Length}_{9\text{feet}} \times \left(\frac{1}{3}H_{gran}\right) = 8.6$

$M_{soil} := M_{sand} + M_{sandgran} + M_{gran} = 112.679$

$M_{lateral_{first}} := \mu \times \text{Weight of block} \times n_{first} \times \left(H_{sand} + H_{gran}\right) \times A_{m_{first}} = 89.378$

$M_{lateral_{second}} := \mu \times \text{Weight of block} \times n_{second} \times \left(H_{sand} + H_{gran}\right) \times A_{m_{second}} = 83.792$

$M_{lateral_{third}} := \mu \times \text{Weight of block} \times n_{third} \times \left(H_{sand} + H_{gran}\right) \times A_{m_{third}} = 55.861$

$M_{lateral} := M_{lateral_{first}} + M_{lateral_{second}} + M_{lateral_{third}} = 229.032$

$M_{vertical_{first}} := \text{Weight of block} \times n_{first} \times (2.5 \times \text{Length}_{3\text{feet}}) = 446.598$

$M_{vertical_{second}} := \text{Weight of block} \times n_{second} \times (1.5 \times \text{Length}_{3\text{feet}}) = 251.212$

$M_{vertical_{third}} := \text{Weight of block} \times n_{third} \times (0.5 \times \text{Length}_{3\text{feet}}) = 55.825$

$M_{vertical} := M_{vertical_{first}} + M_{vertical_{second}} + M_{vertical_{third}} = 753.635$

$M_{concrete} := M_{lateral} + M_{vertical} = 982.667$

$F_{OS} := \frac{M_{concrete}}{M_{soil}} = 8.721$

At the interface between granular A and Olivine sand:

$n_{first} = 9 \quad n_{second} = 9 \quad n_{third} = 6$
\[ \text{Arm}_{\text{first}} = \frac{1}{2} \quad \text{Arm}_{\text{second}} = \frac{1}{2} \quad \text{Arm}_{\text{third}} = \frac{1}{2} \]

\[ M_{\text{soil}} := P_{\text{sand}} \times \text{Length}_{12 \text{feet}} \left( \frac{1}{3} H_{\text{sand}} \right) = 26.02 \]

\[ M_{\text{lateral\_first}} := \mu \times \text{Weight}_{\text{block}} \times n_{\text{first}} \times H_{\text{sand}} \times \text{Arm}_{\text{first}} = 30.165 \]

\[ M_{\text{lateral\_second}} := \mu \times \text{Weight}_{\text{block}} \times n_{\text{second}} \times H_{\text{sand}} \times \text{Arm}_{\text{second}} = 30.165 \]

\[ M_{\text{lateral\_third}} := \mu \times \text{Weight}_{\text{block}} \times n_{\text{third}} \times H_{\text{sand}} \times \text{Arm}_{\text{third}} = 20.11 \]

\[ M_{\text{lateral}} := M_{\text{lateral\_first}} + M_{\text{lateral\_second}} + M_{\text{lateral\_third}} = 80.44 \]

\[ M_{\text{vertical\_first}} := \text{Weight}_{\text{block}} \times n_{\text{first}} \times (2.5 \times \text{Length}_{3 \text{feet}}) = 251.212 \]

\[ M_{\text{vertical\_second}} := \text{Weight}_{\text{block}} \times n_{\text{second}} \times (1.5 \times \text{Length}_{3 \text{feet}}) = 150.727 \]

\[ M_{\text{vertical\_third}} := \text{Weight}_{\text{block}} \times n_{\text{third}} \times (0.5 \times \text{Length}_{3 \text{feet}}) = 33.495 \]

\[ M_{\text{concrete}} := M_{\text{lateral}} + M_{\text{vertical}} = 515.874 \]

\[ \text{FOS} := \frac{M_{\text{concrete}}}{M_{\text{soil}}} = 19.826 \]

**Movable Portion: Against Rotation**

\[ n_{\text{first}} = 9 \quad n_{\text{second}} = 4 \quad n_{\text{third}} = 4 \]

\[ \text{Arm}_{\text{first}} = \frac{1}{2} \quad \text{Arm}_{\text{second}} = \frac{1}{3} \quad \text{Arm}_{\text{third}} = \frac{1}{3} \]

\[ M_{\text{soil}} := P_{\text{sand}} \times \text{Length}_{12 \text{feet}} \left( \frac{1}{3} H_{\text{sand}} \right) = 26.02 \]

\[ M_{\text{lateral\_first}} := \mu \times \text{Weight}_{\text{block}} \times n_{\text{first}} \times H_{\text{sand}} \times \text{Arm}_{\text{first}} = 30.165 \]

\[ M_{\text{lateral\_second}} := \mu \times \text{Weight}_{\text{block}} \times n_{\text{second}} \times H_{\text{sand}} \times \text{Arm}_{\text{second}} = 8.938 \]

\[ M_{\text{lateral\_third}} := \mu \times \text{Weight}_{\text{block}} \times n_{\text{third}} \times H_{\text{sand}} \times \text{Arm}_{\text{third}} = 8.938 \]

\[ M_{\text{lateral}} := M_{\text{lateral\_first}} + M_{\text{lateral\_second}} + M_{\text{lateral\_third}} = 48.041 \]

\[ M_{\text{vertical\_first}} := \text{Weight}_{\text{block}} \times n_{\text{first}} \times (2.5 \times \text{Length}_{3 \text{feet}}) = 251.212 \]

\[ M_{\text{vertical\_second}} := \text{Weight}_{\text{block}} \times n_{\text{second}} \times (1.5 \times \text{Length}_{3 \text{feet}}) = 66.99 \]

\[ M_{\text{vertical\_third}} := \text{Weight}_{\text{block}} \times n_{\text{third}} \times (0.5 \times \text{Length}_{3 \text{feet}}) = 22.33 \]

\[ M_{\text{concrete}} := M_{\text{vertical\_first}} + M_{\text{vertical\_second}} + M_{\text{vertical\_third}} = 340.531 \]

\[ M_{\text{concrete}} := M_{\text{lateral}} + M_{\text{vertical}} = 388.572 \]

\[ \text{FOS} := \frac{M_{\text{concrete}}}{M_{\text{soil}}} = 14.934 \]
D-2 Test chamber concrete block configuration #1

Scenario #1 FOS = 1.27
ORDER:
2 - 6 Foot blocks plus 1 - 6 Foot top block
3 - 3 Foot blocks
Total Cost $1350 taxes and del, incl.

Total Blocks required:
9 Foot blocks = 10
6 Foot blocks = 13
Add 1 6 Foot block plus 1 6 Foot top block to order,
3 Foot Blocks = 10

Main Wall Block layout
East West View / Profile View

Secondary wall block layout
East West View / Profile View

Tertiary wall block layout
East West View / Profile View

9' Blocks = 6
6' Blocks = 4
3' Blocks = 5

9' Blocks = 4
6' Blocks = 5
3' Blocks = 1

9' Blocks = 0
6' Blocks = 3
3' Blocks = 4
D-3 Test chamber concrete block configuration #2

Scenario #2 FOS = 1.41
ORDER:
5 – 6 Foot blocks plus 1 – 6 Foot top block
3 – 3 Foot blocks
Total Cost $1300 taxes and del. Inc.

Total Blocks required:
9 Foot blocks = 10
6 Foot blocks = 16 Add 1 6 Foot block plus 1 6 Foot top block to order.
3 Foot blocks = 10

Main Wall Block layout
East West View / Profile View

Secondary wall block layout
East West View / Profile View

Tertiary wall block layout
East West View / Profile View

9' Blocks = 6
6' Blocks = 4
3' Blocks = 5

9' Blocks = 4
6' Blocks = 5
3' Blocks = 1

9' Blocks = 0
6' Blocks = 3
3' Blocks = 4
Appendix E Olivine sand compaction records

This provides compaction records of all lifts for both burial tests as measured by the three density cups.
Table E-1 Lift #1 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1691</td>
</tr>
<tr>
<td>2</td>
<td>1719</td>
</tr>
<tr>
<td>3</td>
<td>1756</td>
</tr>
</tbody>
</table>

Table E-2 Lift #2 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1730</td>
</tr>
<tr>
<td>2</td>
<td>1673</td>
</tr>
<tr>
<td>3</td>
<td>1737</td>
</tr>
</tbody>
</table>

Table E-3 Lift #3 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1665</td>
</tr>
<tr>
<td>2</td>
<td>1712</td>
</tr>
<tr>
<td>3</td>
<td>1665</td>
</tr>
</tbody>
</table>

Table E-4 Lift #4 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1639</td>
</tr>
<tr>
<td>2</td>
<td>1654</td>
</tr>
<tr>
<td>3</td>
<td>1678</td>
</tr>
</tbody>
</table>

Table E-5 Lift #5 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1678</td>
</tr>
<tr>
<td>2</td>
<td>1673</td>
</tr>
<tr>
<td>3</td>
<td>1750</td>
</tr>
</tbody>
</table>
Table E-6 Lift #6 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1743</td>
</tr>
<tr>
<td>2</td>
<td>1680</td>
</tr>
<tr>
<td>3</td>
<td>1750</td>
</tr>
</tbody>
</table>

Table E-7 Lift #7 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1717</td>
</tr>
<tr>
<td>2</td>
<td>1680</td>
</tr>
<tr>
<td>3</td>
<td>1730</td>
</tr>
</tbody>
</table>

Table E-8 Lift #8 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1697</td>
</tr>
<tr>
<td>2</td>
<td>1667</td>
</tr>
<tr>
<td>3</td>
<td>1737</td>
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</tbody>
</table>

Table E-9 Lift #9 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1574</td>
</tr>
<tr>
<td>2</td>
<td>1550</td>
</tr>
<tr>
<td>3</td>
<td>1580</td>
</tr>
</tbody>
</table>

Table E-10 Lift #10 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1581</td>
</tr>
<tr>
<td>2</td>
<td>1530</td>
</tr>
<tr>
<td>3</td>
<td>1606</td>
</tr>
</tbody>
</table>
Table E-11 Lift #11 compaction for 0.6 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1672</td>
</tr>
<tr>
<td>2</td>
<td>1641</td>
</tr>
<tr>
<td>3</td>
<td>1730</td>
</tr>
</tbody>
</table>

Table E-12 Lift #12 compaction for 0.6 m burial depth

<table>
<thead>
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<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1626</td>
</tr>
<tr>
<td>2</td>
<td>1647</td>
</tr>
<tr>
<td>3</td>
<td>1639</td>
</tr>
</tbody>
</table>

Table E-13 Lift #1 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1776</td>
</tr>
<tr>
<td>2</td>
<td>1784</td>
</tr>
<tr>
<td>3</td>
<td>1665</td>
</tr>
</tbody>
</table>

Table E-14 Lift #2 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1672</td>
</tr>
<tr>
<td>2</td>
<td>1669</td>
</tr>
<tr>
<td>3</td>
<td>1626</td>
</tr>
</tbody>
</table>

Table E-15 Lift #3 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1724</td>
</tr>
<tr>
<td>2</td>
<td>1699</td>
</tr>
<tr>
<td>3</td>
<td>1782</td>
</tr>
</tbody>
</table>
Table E-16 Lift #4 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1743</td>
</tr>
<tr>
<td>2</td>
<td>1673</td>
</tr>
<tr>
<td>3</td>
<td>1697</td>
</tr>
</tbody>
</table>

Table E-17 Lift #5 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1633</td>
</tr>
<tr>
<td>2</td>
<td>1582</td>
</tr>
<tr>
<td>3</td>
<td>1619</td>
</tr>
</tbody>
</table>

Table E-18 Lift #6 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1502</td>
</tr>
<tr>
<td>2</td>
<td>1530</td>
</tr>
<tr>
<td>3</td>
<td>1613</td>
</tr>
</tbody>
</table>

Table E-19 Lift #7 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1658</td>
</tr>
<tr>
<td>2</td>
<td>1621</td>
</tr>
<tr>
<td>3</td>
<td>1658</td>
</tr>
</tbody>
</table>

Table E-20 Lift #8 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1613</td>
</tr>
<tr>
<td>2</td>
<td>1615</td>
</tr>
<tr>
<td>3</td>
<td>1665</td>
</tr>
</tbody>
</table>
Table E-21 Lift #9 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1659</td>
</tr>
<tr>
<td>2</td>
<td>1673</td>
</tr>
<tr>
<td>3</td>
<td>1658</td>
</tr>
</tbody>
</table>

Table E-22 Lift #10 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1652</td>
</tr>
<tr>
<td>2</td>
<td>1608</td>
</tr>
<tr>
<td>3</td>
<td>1711</td>
</tr>
</tbody>
</table>

Table E-23 Lift #11 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1652</td>
</tr>
<tr>
<td>2</td>
<td>1693</td>
</tr>
<tr>
<td>3</td>
<td>1704</td>
</tr>
</tbody>
</table>

Table E-24 Lift #12 compaction for 1.2 m burial depth

<table>
<thead>
<tr>
<th>Density Cup (#)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1652</td>
</tr>
<tr>
<td>2</td>
<td>1647</td>
</tr>
<tr>
<td>3</td>
<td>1658</td>
</tr>
</tbody>
</table>
Appendix F Assorted Pictures

This section contains assorted photographs from the full-scale articulation test, test chamber construction and both tests at two different burial depths.
F.1 Full Scale Articulation Test

Figure F-1-1 Joint #2 directly below hydraulic actuator prior to test

Figure F-1-2 Joint #1 with total station prism

Figure F-1-3 Pipe assembly prior to testing
Figure F-1-4 Joint #2 secured to hydraulic actuator

Figure F-1-5 Joint #1 post test
Figure F-1-6 Joint #2 post test

Figure F-1-7 Joint #3 post test
F.2 Test Chamber Construction

Figure F-2-1 Moving floor frame installation

Figure F-2-2 North jacks connection to moving floor frame
Figure F-2-3 Completed wooden moving floor deck

Figure F-2-4 Completed friction treatment for western and eastern walls
Figure F-2-5 Density cups prior to soil placement
F.3  Full-Scale Test in 1.2 m Burial Depth

Figure F-3-1 Pipe assembly at southern wall prior to burial

Figure F-3-2 Test pipe exhumed post test
Figure F-3-3 Pipe assembly exhumed post test
F.4  Full-Scale Test in 0.6 m Burial Depth

Figure F-4-1 Pipe assembly lowered to burial depth

Figure F-4-2 External stringpot exhumed post test
Figure F-4-3 External stringpot (exhumed) connected to pipe assembly at springline post test