Macro-scale direct shear device for studying the large displacement shear strength of soil-structure interfaces under very low effective stresses

by

Ruslan Shanth Amarasinghe

B.Sc.Engg. (Hons), The University of Peradeniya, Sri Lanka, 2009

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

in

The Faculty of Graduate Studies

(Civil Engineering)

THE UNIVERSITY OF BRITISH COLUMBIA

(Vancouver)

April 2013

© Ruslan Shanth Amarasinghe 2013
Abstract

This thesis describes a new macro-scale test device for assessing the large-displacement soil/solid interface shear strength at very low effective normal stresses (3 kPa to 6 kPa). The testing method arises from a need to obtain the interface friction between soils and epoxy-coated pipes under low effective normal stress levels which is an important consideration in the design of partly buried seabed pipelines. The test device is fundamentally similar to the conventional small-scale direct-shear apparatus except for its large footprint that provides a plan interface shear area of 1.72 m by 1.75 m. The device is designed to impart displacement-controlled interface-shearing at displacement rates ranging from 0.0001 mm/s to 1 mm/s and with the ability to reach a maximum interface shear displacement of 1.2 m. The desired normal stress at the soil/solid interface is obtained using surcharge loads externally applied by means of bulk sand or water masses, or both in certain cases. The device is instrumented with pressure transducers mounted flush with the top surface of the solid test surface for the measurement of pore water pressure at the shear interface, in turn, allowing accurate determination of the effective normal stress at the soil/solid interface during shearing. The key features of this device are described, and the device capabilities are demonstrated by testing three soil types (Fraser-River sand, non-plastic silt, kaolinite) on two test surfaces (mild steel, epoxy-coated mild steel) at effective normal stresses between 3 kPa and 7 kPa. Comparison of the test results with available findings from other devices is used to further confirm the suitability of the device for the intended purpose.
# Table of Contents

Abstract ................................................................. ii

Table of Contents ...................................................... iii

List of Tables ............................................................. vi

List of Figures ............................................................ viii

Acknowledgements ....................................................... xv

1 Introduction ............................................................. 1
   1.1 Purpose of the Research Program ................................. 3
   1.2 Organization of the Thesis ....................................... 4

2 Literature Review ..................................................... 6
   2.1 State of Practice in the Analysis of Soil-Pipe Interaction Forces in Offshore Deep-Water Applications .................. 8
      2.1.1 Lateral shear resistance of soil-pipeline interfaces ........ 10
      2.1.2 Axial shear resistance of soil-pipeline interfaces .......... 13
   2.2 Laboratory Testing of Soil-Solid Interface Shear Strength ... 15
      2.2.1 Direct shear apparatus ...................................... 16
      2.2.2 Direct simple shear apparatus .............................. 19
      2.2.3 Ring shear apparatus ...................................... 21
      2.2.4 Other devices .............................................. 22

3 The Macro-Scale Interface Direct Shear Device ................. 24
   3.1 Overview of Test Device ....................................... 25
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.4</td>
<td>Saturated-Non-Plastic-Silt / Mild-Steel Interface</td>
<td>86</td>
</tr>
<tr>
<td>4.4.1</td>
<td>Details of tests</td>
<td>87</td>
</tr>
<tr>
<td>4.4.2</td>
<td>Test results</td>
<td>89</td>
</tr>
<tr>
<td>4.4.3</td>
<td>Visual observations</td>
<td>97</td>
</tr>
<tr>
<td>4.5</td>
<td>Saturated-Non-Plastic-Silt / Epoxy-Coated-Mild-Steel Interface</td>
<td>101</td>
</tr>
<tr>
<td>4.5.1</td>
<td>Details of test</td>
<td>101</td>
</tr>
<tr>
<td>4.5.2</td>
<td>Test results</td>
<td>103</td>
</tr>
<tr>
<td>4.6</td>
<td>Saturated-Kaolinite-Clay / Mild-Steel Interface</td>
<td>106</td>
</tr>
<tr>
<td>4.6.1</td>
<td>Details of test</td>
<td>106</td>
</tr>
<tr>
<td>4.6.2</td>
<td>Test results</td>
<td>107</td>
</tr>
<tr>
<td>4.6.3</td>
<td>Visual observations</td>
<td>110</td>
</tr>
<tr>
<td>4.7</td>
<td>Saturated-Kaolinite-Clay / Epoxy-Coated-Mild-Steel Interface</td>
<td>111</td>
</tr>
<tr>
<td>4.7.1</td>
<td>Details of test</td>
<td>111</td>
</tr>
<tr>
<td>4.7.2</td>
<td>Test results</td>
<td>113</td>
</tr>
<tr>
<td>4.8</td>
<td>Conclusion</td>
<td>115</td>
</tr>
<tr>
<td>Bibliography</td>
<td></td>
<td>122</td>
</tr>
</tbody>
</table>
## List of Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Examples of lateral resistance models ([Cathie et al.] 2005)</td>
<td>12</td>
</tr>
<tr>
<td>2.2</td>
<td>Summary of previous research on soil-solid interfaces conducted using the direct shear apparatus</td>
<td>18</td>
</tr>
<tr>
<td>2.3</td>
<td>Summary of previous research on soil-solid interfaces conducted using the direct simple shear apparatus</td>
<td>20</td>
</tr>
<tr>
<td>2.4</td>
<td>Summary of previous research on soil-solid interfaces conducted using the ring-shear apparatus</td>
<td>22</td>
</tr>
<tr>
<td>4.1</td>
<td>Chemical constituents of the kaolinite clay test-soil.</td>
<td>64</td>
</tr>
<tr>
<td>4.2</td>
<td>Summary of interface direct shear tests conducted on the new device.</td>
<td>66</td>
</tr>
<tr>
<td>4.3</td>
<td>Details of dry-Fraser-River-sand / mild-steel interface direct shear tests conducted on the macro-scale interface direct shear device.</td>
<td>68</td>
</tr>
<tr>
<td>4.4</td>
<td>Test results of dry-Fraser-River-sand / mild-steel interface direct shear tests conducted on the macro-scale interface direct shear device.</td>
<td>77</td>
</tr>
<tr>
<td>4.5</td>
<td>Test results of dry-Fraser-River-sand / mild-steel interface direct shear tests conducted on the conventional direct shear device.</td>
<td>80</td>
</tr>
<tr>
<td>4.6</td>
<td>Results of dry-Fraser-River-sand / mild-steel interface direct shear tests conducted on the conventional direct shear device and the macro-scale interface direct shear device compared.</td>
<td>83</td>
</tr>
<tr>
<td>4.7</td>
<td>Test results of coarse-grained-soil / mild-steel interface shear tests conducted on conventional interface shear apparatus compared with the macro-scale interface direct shear results.</td>
<td>85</td>
</tr>
<tr>
<td>4.8</td>
<td>Details of saturated-non-plastic-silt/mild-steel interface direct shear tests conducted on the macro-scale interface direct shear device.</td>
<td>89</td>
</tr>
</tbody>
</table>
4.9  Test results of macro-scale interface direct shear tests conducted on saturated-non-plastic-silt/mild-steel interface. ........................................... 96
4.10 Comparison of macro-scale soil/solid interface direct shear test results of interfaces tested. ................................................................. 116
## List of Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Bi-linear model for lateral pipe-soil interaction.</td>
<td>11</td>
</tr>
<tr>
<td>2.2</td>
<td>Typical configurations of the modified direct shear apparatus used for</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>interface shear testing.</td>
<td></td>
</tr>
<tr>
<td>2.3</td>
<td>Typical configuration of the simple shear type apparatus used for soil-</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>solid interface shear testing.</td>
<td></td>
</tr>
<tr>
<td>3.1</td>
<td>Schematic diagram of the macro-scale interface shear test device and test</td>
<td>26</td>
</tr>
<tr>
<td></td>
<td>setup.</td>
<td></td>
</tr>
<tr>
<td>3.2</td>
<td>Photograph of the macro-scale interface shear test device.</td>
<td>27</td>
</tr>
<tr>
<td>3.3</td>
<td>Plan and end elevation of the stationary base.</td>
<td>29</td>
</tr>
<tr>
<td>3.4</td>
<td>Photograph of test plates attached to the stationary base showing special</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>distribution of pore-water pressure sensor apertures.</td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td>Plan view and end elevation of the mobile frame.</td>
<td>32</td>
</tr>
<tr>
<td>3.6</td>
<td>Details of rubber wiper used to seal the gap between the mobile frame and</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>test surface. (Cross-section of the North end of the mobile frame shown).</td>
<td></td>
</tr>
<tr>
<td>3.7</td>
<td>Photograph of the mobile frame.</td>
<td>34</td>
</tr>
<tr>
<td>3.8</td>
<td>Photograph of the North side of the device showing the load cell,</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>string-potentiometer, stepper motor, and the worm-and-gear mechanism.</td>
<td></td>
</tr>
<tr>
<td>3.9</td>
<td>Photograph illustrating the hinged connection between the load cell and</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td>the threaded rod transferring the tensile force to the mobile frame.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(Note that the hinge shown, is in a state with no load applied hence the</td>
<td></td>
</tr>
<tr>
<td></td>
<td>slack at the hinge in the photograph).</td>
<td></td>
</tr>
<tr>
<td>3.10</td>
<td>Details of pin connection between the load cell and the threaded rod.</td>
<td>37</td>
</tr>
</tbody>
</table>
### List of Figures

3.11 Photograph of pressure transducer that is used for pore-water pressure measurement. .................................................. 38

3.12 Details of pressure transducer connection to the test plate. ........ 39

3.13 Details of pressure transducer saturation procedure. (a) Saturation of pressure transducer cavity. (b) Attachment of saturated transducer to stainless steel adapter and saturation of adapter. (c) Placement of porous stone. .................................................. 40

3.14 Schematic cross-section showing test specimen. Surcharge sand having a constant thickness placed over the test soil/base-plate interface. (Not to scale). .................................................. 43

3.15 Placement of geo-cell grid on top of the soil specimen in preparation of placement of surcharge load sand layer. (a) Soil specimen lined with the geotextile filter layer. (b) Geo-cell confinement grid placed on top of the test specimen and ready to be filled with sand. ............. 45

3.16 Surcharge load sand layer reinforced with geo-cell grid prepared on top of the soil specimen. .................................................. 46

3.17 Preparation of side-walls of the mobile frame for receiving soil specimen. (a) Non-woven geotextile side-skirts attached to interior of the aluminium mobile frame. (b) Side-walls of the mobile frame lined with two layers of 6-mil polyethylene sheeting. .................................................. 49

3.18 Coarse-grained soil specimen placed inside the mobile frame. ...... 50

3.19 Plastic container of 124 L capacity and the electric mixer used in the preparation of the slurry. .................................................. 52

3.20 Preparation of the fine-grained soil slurry. .................................................. 53

3.21 Fine-grained soil specimen after placing inside the mobile frame. ... 54

3.22 Typical pore-water pressure dissipation profile during consolidation of fine-grained soil specimens under surcharge loading measured on the macro-scale interface direct shear device. .................................................. 55

3.23 Diagram showing the self-weight settled soil specimen and the clear water surface formed during settlement. ........................ 55
3.24 Pore-water pressure dissipation profile obtained during consolidation of fine-grained soil specimen used to calculate the coefficient of one-dimensional consolidation of the soil specimen. .......................... 57

3.25 Schematic free body diagram of the mobile frame of the macro-scale interface direct shear test device. ................................. 59

4.1 Grain size distribution of soils tested. ................................. 62

4.2 Small coupon of mild-steel test surface used in the macro-scale interface direct shear tests. .................................................. 64

4.3 Variation of device friction with shear displacement measured by displacing the mobile frame with no soil specimen inside the mobile frame. (Device frictional resistance is the ratio of device friction force $F_{df}$ to interface shear area $A_i$). .................................................. 69

4.4 Photograph showing the components of the conventional direct shear apparatus used in undergraduate level laboratory classes at UBC. 70

4.5 Photograph showing the modified components of the conventional direct shear apparatus. .................................................. 71

4.6 Photograph showing the modified conventional direct shear apparatus that was used for interface shear testing. ................................. 72

4.7 Photograph showing the dry Fraser River sand specimen prepared inside the top shear frame of the modified conventional direct shear apparatus. .................................................. 73

4.8 Photograph showing the top cap placed on top of the dry Fraser River sand specimen. .................................................. 73

4.9 Photograph showing the surcharge load application mechanism on the modified conventional direct shear apparatus. ................................. 74

4.10 Fraser-River-sand/mild-steel macro-scale interface direct shear test results. (a) Variation of average shear stress with shear displacement. (b) Variation of normalized shear resistance with shear displacement. 75
4.11 Mean interface shear stress at large displacement versus average effective normal stress obtained from Fraser-River-sand/mild-steel macro-scale interface direct shear tests. 

4.12 Photograph showing the South end of the macro-scale interface direct shear device at the end of a Fraser-River-sand/mild-steel interface shear test.

4.13 Photograph showing the top of the mobile frame at the end of a Fraser-River-sand/mild-steel interface shear test.

4.14 Variation of average interface shear stress with shear displacement for dry-Draser-River-sand/mild-steel interface obtained from the conventional direct shear device.

4.15 Variation of normalized interface shear stress with percentage shear displacement for dry-Draser-River-sand/mild-steel interface obtained from the conventional direct shear device and the macro-scale interface direct shear device.

4.16 Typical variation of average pore-water pressure at the test surface level observed during preparation of a silt specimen and surcharge loading.

4.17 Test results of macro-scale interface direct shear tests on saturated non-plastic silt against a mild-steel test surface. (a) Variation of average pore-water pressure at the interface with shear displacement. (b) Variation of average effective normal stress at the interface with shear displacement.

4.18 Variation of pore-water pressure with shear displacement observed during macro-scale interface direct shear tests on saturated non-plastic silt against a mild-steel test surface conducted on May/12/2012.

4.19 Variation of pore-water pressure with shear displacement observed during macro-scale interface direct shear tests on saturated non-plastic silt against a mild-steel test surface conducted on May/16/2012.
4.20 Variation of pore-water pressure with shear displacement observed during macro-scale interface direct shear tests on saturated non-plastic silt against a mild-steel test surface conducted on May/09/2012. 

4.21 Test results of macro-scale interface direct shear tests on saturated non-plastic silt against a mild-steel test surface. (a) Variation of average shear stress with shear displacement. (b) Variation of normalized shear resistance with shear displacement.

4.22 Mean interface shear stress at large displacement versus average effective normal stress obtained from saturated-non-plastic-silt/mild-steel macro-scale interface direct shear tests.

4.23 Variation of normalized interface shear resistance with shear displacement obtained from macro-scale interface direct shear tests on saturated-non-plastic-silt/mild-steel and dry-Fraser-River-sand/mild-steel interfaces compared.

4.24 Photograph showing the South end of the macro-scale interface direct shear device at the end of a non-plastic-silt/mild-steel interface direct shear test.

4.25 Photograph showing the top of the mobile frame at the end of a non-plastic-silt/mild-steel interface direct shear test after carefully removing the surcharge load layers to expose the silt specimen.

4.26 Photograph showing the depression of the silt specimen at the North edge observed at the end of a non-plastic-silt/mild-steel interface direct shear test after carefully removing the surcharge load layers to expose the silt specimen.

4.27 Variation of pore-water pressure at the test surface observed during preparation of the silt specimen on the epoxy-coated mild steel surface.

4.28 Variation of pore-water pressure at the soil/solid interface with shear displacement observed during the non-plastic-silt/epoxy-coated-mild-steel interface direct shear test conducted on the macro-scale device.
4.29 Test results of the non-plastic-silt/epoxy-coated-mild-steel interface direct shear test conducted on the macro-scale device. (a) Variation of the average interface shear stress with shear displacement. (b) Variation of the normalized interface shear resistance with shear displacement. .................................................. 105

4.30 Variation of average pore-water pressure at the test surface level observed during preparation of kaolinite clay specimen and surcharge loading. ................................................................. 107

4.31 Test results of macro-scale interface direct shear test on saturated kaolinite clay against a mild-steel test surface. (a) Variation of average pore-water pressure at the interface with shear displacement. (b) Variation of average effective normal stress at the interface with shear displacement. .................................................. 108

4.32 Test results of macro-scale interface direct shear test on saturated kaolinite clay against a mild-steel test surface. (a) Variation of average shear stress with shear displacement. (b) Variation of normalized shear resistance with shear displacement. .................................................. 109

4.33 Photograph showing the top of the mobile frame at the end of the kaolinite-clay/mild-steel interface direct shear test after carefully removing the surcharge load layers to expose the clay specimen. ................................................. 110

4.34 Time history of the recorded pore water pressure at the base of the test soil during application of surcharge load and subsequent consolidation. 112

4.35 Variation of pore-water pressure at the soil/solid interface with shear displacement observed during the kaolinite-clay/epoxy-coated-mild-steel interface direct shear test conducted on the macro-scale device. ... 114

4.36 Test results of the kaolinite-clay/epoxy-coated-mild-steel interface direct shear test conducted on the macro-scale device. (a) Variation of the average interface shear stress with shear displacement. (b) Variation of the normalized interface shear resistance with shear displacement. .................................................. 115
List of Figures

4.37 Variation of normalized interface shear resistance with shear displacement observed for some of the soil/solid interfaces tested on the macroscale device. .................................................. 117
Acknowledgements

My gratitude goes out to my supervisor, Dr. Dharma Wijewickreme, who has provided me with support and guidance throughout this endeavour. Be it at night or day, weekday or the weekend, he was always available to consult with whenever I had the need to. His encouragement, guidance, and invaluable advice has helped me immensely, not only to complete this research, but also to develop myself as a professional in the community. I am truly indebted and thankful to him for his moral support and the time he devoted for myself.

I would like to thank my wife, Anupama, for her patience, love, and constant support. Her commitments towards her PhD at the University of Manitoba and mine at UBC has kept us geographically separated for the duration of this work and it has been difficult on her and myself. My work here at UBC would not have been possible without her love and support. Thanks will not be enough for my parents from whom I have been constantly away.

I am grateful to Dr. Yogi Vaid, Dr. John Howie, Dr. Mahdi Taiebat, Dr. Jonathan Fannin, and Dr. Noboru Yonemitsu of the Department of Civil Engineering of UBC for their invaluable advice and support. I would also like to thank all faculty members for providing an exceptional academic learning experience. Special thanks to Mark Rigolo, Director of Laboratories at the Department of Civil Engineering of UBC for his help and support throughout this research program. I would also like to thank all the staff members of the Department of Civil Engineering of UBC for their help.

Special thanks to my friends and colleagues Gaziz Siedalinov, Ainur Siedalinova, Anton Dabeet, Lalinda Weerasekara, Buddhika Samarakoon, Gamini Siriwardana, Lekha Samanmalee, Anajana Punchihewa, Emalayan Vairavanathan, Primal Wijesekera, and Nuwan Devapriya for their support. Thanks to all my colleagues and
senior graduate students of the Geotechnical Group of the Department of Civil Engineering for their valuable comments and moral support.

The funding provided by the Qatar National Research Fund (QNRF), Qatar, under Project No: NPRP 08 2032064, for this research work is deeply appreciated. Thanks are also due to Emilie Lapointe, P.Eng, M.A.Sc., Construction Engineer at Golder Construction Inc., M.A.Sc. student Xavier Tellier, Undergraduate Student Research Assistants Ms. Della Anggabrata, Mr. Elliot Yii, Mr. Norman Richardson, Mr. Daniel Fortin, Mr. Michael Ross Ang, Mr. Eric Tam and Graduate Student Laboratory Assistants Mr. Mehrdad Mirhossaini, Mr. Ji Lv and Mr. Mark Yang for their contributions during laboratory testing and manuscript preparation. Technical assistance of Messrs. Harald Schrempp, Bill Leung, Scott Jackson and John Wong of the Department of Civil Engineering Workshop is also deeply appreciated.
Chapter 1

Introduction

A major consideration in the design of deep water pipelines is the need to evaluate their performance when subjected to potential differential movements due to thermal expansion or contraction under successive start up and shut down cycles. Such differential pipeline movements are typically associated with large strains at the pipe/soil interface. Since a very large proportion of offshore pipelines are located essentially at the seabed level, these interface shear strains are developed under very low effective normal stress levels. For most seabed soil types, development of such strains can significantly reduce the pipe/soil interface strength to its residual value and consequently affect the stability of pipelines placed on sloping sea floors and the potential of pipeline walking.

The pipe/soil interface shear strength under low effective normal stress levels is one of the important input parameters in pipe/soil interaction models used in the design of seabed pipelines. However, there is much uncertainty involved in determining appropriate design values for this parameter due to the lack of experimental data currently available. The effective normal stresses experienced by submarine pipelines lying on the seabed are generally less than 6 kPa. The usual approach to this problem has been to rely on data obtained from conventional geotechnical apparatus such as the ring-shear and direct shear devices. However, it is difficult (if not impossible) to use these devices to capture the soil-solid interface friction angle under very low effective normal stresses. Problems are often attributed to the mechanical friction related errors that are emphasized at such low stress levels, limited displacement ranges, and difficulty in measuring excess pore-water pressure generation during shear. Testing for the interface shear strength under effective normal stresses greater than 50 kPa are quite common on these apparatus and are often used in pile foundation design.
applications. In fact, most soil-pipeline design methodologies stem from experience derived from research and case studies of pile foundations applications. However it is often found that these methods are not always directly applicable to offshore pipeline design and often require modifications in order to be of use in this specialized application.

Also, there is a certain degree of doubt whether the phenomenon observed in these different situations would be the same or in some way different to that observed under relatively high effective normal stress levels, because in offshore deep-water environments the effective normal stresses acting at the soil-pipeline interface is several magnitudes lower than that experienced in typical geotechnical applications. There is evidence to support that the peak internal friction angle of soils may depend on the effective confining stresses when such low confining stresses are considered (Bolton, 1986; Chakraborty and Salgado, 2010; Fannin et al., 2005; Sture et al., 1998). There is further evidence suggesting that the interface shear strength for certain soil-solid interfaces is dependent on the effective normal stress at very low effective normal stress levels (Pedersen et al., 2003). There is also uncertainty about the nature of the shearing mechanism at the soil-solid interface at low effective normal stresses versus at high effective normal stresses. Attempts have been made to develop novel apparatus that are specifically used to experimentally measure the soil-solid interface shear strength under very low effective normal stresses. The Cam-shear apparatus developed at Cambridge University in UK (Kuo and Bolton, 2009) and the Tilt-Table apparatus (Najjar et al., 2007) can be cited as examples of such apparatus.

For the above reasons, it was identified that there is a need to develop a suitable method for the experimental investigation of drained shear strength characteristics at soil-solid interfaces under:

- Very low effective normal stresses (typically in the range 3 kPa to 6 kPa);
- and at relatively large shear displacements

and this need is addressed by the experimental work undertaken in this research. In particular, the development, design, fabrication, and verification of a new macro-scale
interface direct shear test device capable of performing soil-solid interface direct shear tests under low stresses formed the main theme of this thesis.

The research work was undertaken as a collaboration between Qatar University, Doha, Qatar and the University of British Columbia, Vancouver, B.C., Canada and with funding from the Qatar National Research Fund (QNRF). The main purpose of the research program as well as a summary of the content covered in this thesis are presented, in respective order, in the next two sections.

1.1 Purpose of the Research Program

The main purpose of this research program is to produce experimental data and new insight into the problem of soil-pipeline interaction in offshore environments under loading conditions specific to offshore oil and gas pipelines. Primarily the research is divided into three components: (i) large-scale laboratory testing of soil-pipeline interfaces under very low effective normal stresses; (ii) large-scale laboratory testing of soil-pipe interaction; and (iii) small-scale element level laboratory testing of soil-pipeline interfaces. The first two parts of the research are conducted at the University of British Columbia whereas the third is to be conducted at Qatar University. It is expected that this research will contribute immensely to the development of the offshore geotechnical engineering profession. This thesis contributes to the Part (i) above of the overall research program. The primary goals of the study that was undertaken are as follows:

1. Design and fabrication of a novel macro-scale interface direct shear test device capable of performing soil-solid interface shear tests under the following conditions:

   • should be capable of testing fine-grained or coarse-grained soils against solid surfaces commonly used in the industry;
   • displacement-controlled shearing done under a constant shear displacement rate;
1.2. Organization of the Thesis

- should be capable of a maximum shear displacement of over 0.75 m;
- very low shear displacement rates are to be applied to ensure full-drainage conditions within the soil specimen when fine-grained soils are tested;
- should provide means of monitoring excess pore-water pressure variation during shearing.

A macro scale interface direct-shear testing device having a plan interface shear area of 1.72 m by 1.75 m was newly developed for this purpose.

2. Conducting of interface shear tests to evaluate the performance of the new device.

The experimental research program involved testing the large-displacement interface friction of a number of soil/solid interfaces under very low effective normal stresses. In summary the following interfaces were tested:

1. Fraser-River sand on a mild steel surface.
2. Non-plastic silt on the same mild steel surface.
3. Kaolinite on the same mild steel surface.
5. Kaolinite on the same epoxy-coated mild steel surface.

1.2 Organization of the Thesis

Chapter I presents an introduction to the thesis. An outline of the work that was carried out in the research program and the basis behind this work is presented.

Chapter II presents a literature review covering important geotechnical aspects of offshore pipeline design and different methods in use for the determination of the soil-pipeline interface shear strength. The chapter is aimed at demonstrating the need to
1.2. Organization of the Thesis

develop new methods specifically applicable in offshore oil and gas pipeline design. An overview of conventional interface shear test devices is presented and their limitations with respect to their use in this specialized field of geotechnics are highlighted.

Chapter III presents details of the newly developed macro-scale interface direct shear device. Principle of operation of the device, function of each component of the device and its important features are described. The chapter is concluded with a detailed description of how each type of soil specimen is prepared for testing.

Chapter IV presents details of the macro-scale interface shear tests that were conducted on the new device in order to evaluate the performance. The results obtained from the new device are compared with those obtained from conventional apparatus. Interface shear tests obtained from conventional direct shear tests, ring shear tests, and tilt table tests are compared. The chapter is concluded with a discussion on the performance of the new device and provides recommendations for future improvement of the device.
Chapter 2

Literature Review

Commonly available direct-shear and ring-shear devices do not always serve as the best option when testing for the soil-solid interface shear strength at a suitable accuracy when very low effective normal stresses are used (Fang et al., 2004). This is mainly because of the relatively small shear area available combined with the friction associated with the mechanical system in these devices. Due to low interface shear strengths expected under small confining stress levels, shear forces measured from small-scale direct shear tests would be relatively small and would also be affected by the friction associated with the mechanical components of the devices (Lehane and Liu, 2013). As such, special modifications are necessary for accurately and reliably testing for the soil/solid interface shear strength under very low confining stresses in such apparatus. For example, the electrical background noise in the data acquisition system of these devices is often at the same order of magnitude as the electrical signals generated by the load cells that measure shear loads at very low confining stresses (Likos et al., 2010). Use of precision instrumentation and data acquisition systems is required in order to ensure that reliable measurements are made. In addition, stress non-uniformities are quite common due to the manner in which the normal stress is applied in most of these devices (Hsieh and Hsieh, 2003; Lings and Dietz, 2004; OSullivan et al., 2004). Moreover, the use of the direct-shear device often requires multiple reversals of the shear direction to arrive at the large displacement friction angle, and this does not necessarily represent field conditions (Mesri and Huvaj-Sarihan, 2012). Another limitation of these devices is the difficulty to perform interface shear tests with reliable excess pore-water pressure measurements to ensure drained conditions during shearing of fully saturated fine-grained soils. The use of pore-water pressure transducers at the soil/solid interface is required in order to measure the pore-water
pressure variation at the interface during shear so that an accurate determination of
the effective normal stress can be made (Fox and Stark, 2004). When testing under
small effective stress levels, the stress distribution at the interface can also be affected
by the physical boundaries of these devices and this can lead to significant errors in
determining the interface shear resistance. Hence the use of conventional appara-
tus for studying the soil/solid interface shear strength at large shear strains under
very low effective normal stresses may not always be suitable, unless the devices are
modified to minimize such uncertainties.

Despite such limitations, traditional testing methods are still quite useful and they
are being used to study soil/solid interface shear strength at relatively low effective
normal stresses (2.4 kPa to 70 kPa). This is particularly evident in work involv-
ing studying geosynthetic clay liners and shallow slope failures where such effective
stresses are expected at the shearing zone. The direct shear device has been used
to test for the internal shear strength of geosynthetic clay liners under applied to-
tal normal stresses ranging from 2.4 kPa to 70 kPa (Gilbert et al., 1996; Zornberg
et al., 2005). Because of the difficulties in ascertaining the pore-water pressure at the
soil/solid interface during shearing, the results derived from such apparatus are often
questionable when fine-grained soils are tested (Bemben and Schulze, 1997). Since
the conventional direct shear device provides a limited shear displacement range it
often fails to capture the large-displacement interface shear strength (Gilbert et al.,
1996). The ring shear device is also being used for the purpose of characterizing
soil/solid interface shear behaviour in the specialized field of geosynthetics. Test-
ing at total normal stresses as low as 50 kPa have been conducted in the past to
study the shear strength of various soil/geosynthetic interfaces using the ring-shear
apparatus (Effendi, 1995). Limitations arising from conventional devices to study the
large-displacement interface shear strength of soil/solid interfaces under very low ef-
efFective normal stresses (i.e., 3 to 6 kPa) have prompted the development of a new
device undertaken in the research program presented herein.

It is also important to note that with particular reference to the offshore pipeline
development applications where the pipe/soil interface shear strength at low effec-
2.1 State of Practice in the Analysis of Soil-Pipe Interaction Forces in Offshore Deep-Water Applications

Pipelines used for the transportation of oil, gas, and liquid fuels across the seabed at depths exceeding 2 km below the water surface of the ocean are often installed by simply laying the pipeline on the seabed - burying or restraining the pipeline at these depths is often an infeasible option (Bruton et al., 2006; Cathie et al., 2005; Randolph et al., 2011). Typically, problems arise when (i) the pipeline crosses some unstable area such as an active fault, a slow moving downhill landslide, area susceptible to liquefaction etc.; and (ii) the pipeline material undergoes expansion or contraction under...
successive start-up and shut-down cycles which leads to longitudinal displacements and lateral buckling of the pipeline. These can significantly affect the pipeline causing large strains and hence high stresses which can lead to the failure of the pipeline (Cathie et al., 2005; Nielsen et al., 1990; Perinet and Simon, 2011). The correct design of the pipeline layout can often avoid scenarios where large differential movements of the pipeline with respect to the soil occur, but certain situations can arise where this is not possible. Moreover, when large spans are involved, the temperature and pressure induced expansion or contraction of the pipeline material can lead to significantly large longitudinal as well as lateral displacements of the pipeline. Hence, the design of such unrestrained seabed pipelines brings forth the need to account for the ways in which the pipeline interacts with the soil.

Pipeline design and engineering will require geotechnical data if major assumptions are to be avoided. Currently there is a wide range of methods that can be used to adequately characterize the geotechnical properties of soils. In offshore pipeline applications involving soft soils this is often accomplished with box corers coupled with in-situ vane testing (OSIF, 1999; Randolph and Gourvenec, 2011). The cone penetration test (CPT) and the full-flow (T-bar or ball) penetration test may also be viable options (Randolph and Gourvenec, 2011). While a geotechnical site investigation provides highly useful information about the soil material for design purposes, additionally, knowledge of the soil-pipeline interaction phenomenon as well as mathematical models that describes the phenomenon in reasonable accuracy is required in order to predict the forces acting on a pipeline subject to differential shear at the soil-pipeline interface. Most mathematical models currently in use stem from experience of studies on the skin friction of piles and are mostly semi-empirical. These models are not always applicable to soil-pipe interaction problems since the loading conditions at the soil-pipeline interface as well as the direction and magnitude of the interfacial shear strains observed are often very much different from that of conventional piles. Differences are also attributed to the relatively infinite length of the pipeline and the non-axisymmetric nature of the problem (Cathie et al., 2005). There is generally a lack of availability of design tools backed by experimental evidence when it comes to
large-scale deep-water oil and gas projects. Laboratory testing in many ways assist understanding complex mechanisms prevalent in soil-pipeline interaction scenarios. Especially this is applicable to offshore pipeline design projects where the soil is mainly fine-grained and effective-stress-based analysis tools are not yet developed to full potential (Brennoddon and Stokkeland, 1992; Bruton et al., 2006; Dendani and Jaeck, 2007; Oliveira et al., 2005). A brief account of the models currently used for determining the soil-pipeline lateral and axial resistance is presented in the next two sub-sections.

2.1.1 Lateral shear resistance of soil-pipeline interfaces

There are currently three different approaches in use for predicting the lateral shear resistance of soil-pipeline interfaces (Cathie et al., 2005):

1. a single friction factor approach where the lateral resistance is related to the submerged weight of the pipeline and the soil type;

2. a two component model consisting of a sliding resistance component and a lateral passive pressure component frictional model supplemented with passive resistance of a wedge of soil (Lieng et al., 1988; Nyman 1984; Verley and Lund 1995; Verley and Sotberg 1992; Wagner et al. 1987);

3. plasticity model approach (Zhang et al. 1999, 2002).

The traditional design approach has been to assume frictional behaviour, such that, the limiting horizontal force on a given length of the pipeline is proportional to the weight of the pipe segment as given by Eq. [2.1]

\[ \frac{H_{\text{max}}}{V} = \mu \]  

Here, \( H_{\text{max}} \) is the maximum horizontal force that a pipe experiences during lateral displacement on the seabed, and \( V \) is the vertical load at the soil-pipe interface. The friction factor \( \mu \) is the coefficient of friction for the soil-pipe interface and ranges
in value from 0.2 to 0.8 (White and Cheuk, 2008). The definition of \( H_{\text{max}} \) stems from conventional bi-linear (linear-elastic perfectly plastic response) models for lateral resistance where the interaction between the pipe and the seabed is modelled by spring-slider elements at intervals along the pipe. The resulting bi-linear response is shown in Fig. 2.1. Parameters \( u \) and \( D \) in Fig. 2.1 are the lateral displacement and diameter of the pipe respectively. The theoretical basis for the use of a friction factor for the soil-pipeline lateral resistance seems to originate from early work on dry friction in the mechanical engineering sciences. Further details on this can be found in (Cathie et al., 2005; White and Cathie, 2010; White and Cheuk, 2008; Zhang et al., 1999, 2002), and therefore, are not presented in this text for brevity.

![Bi-linear model for lateral pipe-soil interaction.](image)

Figure 2.1: Bi-linear model for lateral pipe-soil interaction.

A short list of equations associated with two component models commonly used for the determination of the lateral resistance is shown in Table 2.1. Most of these models are primarily based on empirical data fitting of laboratory test data. These may not always be useful, especially in offshore deep-water applications where large displacements of the pipeline may be involved and when the soil-pipeline interaction is to be modelled using finite element methods.

The plasticity model approach presented by (Zhang et al., 2002) attempts to provide a constitutive model for the soil-pipe interaction problem in Calcareous sand where the effect of pipe geometry, pipe embedment and soil-pipe interface shear strength at very low normal stresses are taken into account. The model needs to be calibrated using soil-pipe interaction tests conducted under laboratory conditions.
### Table 2.1: Examples of lateral resistance models (Cathie et al., 2005)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Equations</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wagner et al. (1987)</td>
<td>$F_y = \mu(W' - F_L) + \beta \gamma' A$</td>
<td>$\gamma' &lt; 8.6kN/m^3$ Monotonic $\mu = 0.6, \beta = 38$</td>
</tr>
<tr>
<td></td>
<td>$F_y = \text{horizontal resistance}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A = 0.5 \times \text{embedded area}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$W' = \text{submerged pipe weight}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_L = \text{hydrodynamic lift}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\gamma' &gt; 9.6kN/m^3$ Monotonic $\mu = 0.6, \beta = 79$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cyclic load $\gamma' &lt; 8.6kN/m^3$ Embedded $\times 2$ $\beta$ reduced by 50%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cyclic load $\gamma' &gt; 9.6kN/m^3$ Embedded $\times 3$ $\beta$ reduced by 80-90%</td>
<td></td>
</tr>
<tr>
<td>Lieng et al. (1988)</td>
<td>$F_y = \mu(W - F_L) + F_R$ $\mu = 0.2, \beta = 39.7$ Cyclic $\gamma' &lt; 70 kPa$ $\mu = 0.6$ (sands) $\mu = 0.2$ (clays) $F_R$ calculated considering accumulated energy $\beta = 15.7$</td>
<td></td>
</tr>
<tr>
<td>Verley and Sotberg (1992)</td>
<td>$F_y = F_c + F_R$ $\mu = 0.6$ All sands</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_y = \mu(W' - F_L) + F_R$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_R = \gamma'D^2(4.5 - 0.11\gamma'D^2/F_c)(z/D)^{1.25}$</td>
<td></td>
</tr>
<tr>
<td>Verley and Lund (1995)</td>
<td>$F_y = F_c + F_R$ $\mu = 0.2$ Clays</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_y = \mu(W' - F_L) + F_R$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_R = 4.13DS_u[S_u(\gamma D)]^{-0.392}(z/D)^{1.31}$ $\mu = 0.2$</td>
<td></td>
</tr>
</tbody>
</table>
2.1.2 Axial shear resistance of soil-pipeline interfaces

The $\alpha$ method, which is based on total stress analysis, has been extensively used in the past for determining axial shear resistance of soil-pipeline interfaces especially when the rate of shear displacement of the pipeline provides little time for good drainage conditions in fine-grained soil (Randolph and Gourvenec, 2011; White and Cathie, 2010). This is an empirical method that stems from experience in pile design where the shear strength at the soil-pipeline interface is given as:

$$F_x = \alpha s_u L$$

(2.2)

where, $s_u$ is the undrained shear strength of the soil, $\alpha$ - referred to as the adhesion factor - is a parameter that takes into account many physical factors and disturbances in the soil, and $L$ is the arc length in embedded soil (including heave). Appropriate values of the shear strength and adhesion factor will depend on whether the peak or residual axial resistance is required, and how long the pipeline has been installed without load. Laboratory shear tests are recommended for the specific soil and coating under consideration. In very soft clays, in the absence of specific data, and subject to the roughness of the interface, the adhesion factor may be taken as 1 for the peak resistance and related to the soil sensitivity, $S_t$, for the residual strength ($\alpha = 1/S_t$) (Cathie et al., 2005). In cases where freely draining soils are involved or when the shearing rate is known to be slow enough for drained conditions to dominate, the $\beta$ method is adopted where simple Coulomb friction models are used to evaluate the axial resistance of partially embedded pipelines. In this case the axial shear strength at the soil-pipeline interface is given as:

$$F_x = \mu W'$$

(2.3)

where, $W'$ is the submerged weight of the pipe and $\mu$ is the coefficient of friction for the soil-pipeline interface. Various guidelines are available for the selection of an appropriate value for $\mu$ but are not yet comprehensive enough to cover all possible
design situations. Three such guidelines are as follows:

1. (RP2A-WSD, 2000)
   \[ \mu = \tan(\phi' - 5^\circ) \]

2. (B.S.8010:part3, 1993)
   - Non-cohesive soil: \( \mu = 0.55 - 1.2 \),
   - Cohesive soil: \( \mu = 0.3 - 1.0 \)

3. (Bruton et al., 2006) - For fully drained conditions \( \mu_{\text{axial}} = \tan(\delta) \), where \( \delta \) is the angle of friction for the soil-pipe interface.

The ability to determine \( \mu \) for a given interface to a reasonable degree of accuracy is critical to effectively representing in-situ conditions with respect to a seabed pipeline. In particular, there is a need to determine the coefficient of friction for a given soil-pipe interface at very low effective normal stresses in the laboratory. The following section briefly documents the various apparatus conventionally used for the study of soil-solid interfaces and attempts to show the limitations of each device in its applicability in offshore pipeline design.
2.2 Laboratory Testing of Soil-Solid Interface Shear Strength

Interface characteristics between various soils and construction materials in geotechnical applications has been a widely studied topic especially over the last 30 years. The earliest of studies in this area of expertise have been focused on understanding the fundamentals of shaft friction, also referred to as skin friction, in pile foundations engineering. Interfaces involving various soils and common construction materials such as steel, timber and concrete have been extensively studied (Bosscher and Ortiz, 1987; Brumund and Leonards, 1973; Fakharian and Evgin, 1997; Potyondy, 1961; Reddy et al., 2000). Similar studies have been reported concerning the study of soil-solid interfaces for the design of soil reinforcement. The behaviour of interfaces between various soils and geosynthetics has also been given considerable attention with the expansion in the use of geosynthetics in industry for geotechnical construction purposes (Frost and Han, 1999; Negussey et al., 1989; O’Rourke et al., 1990; Stark and Poeppel, 1994). Recently, with the development of the offshore oil and gas industry, the development of subterranean and submerged pipelines that traverse vast distances across continents has also presented new areas of study directly related to soil-solid interfaces. The endeavours undertaken in these various fields of research have produced a variety of tools and methods that are commonly used to determine the shear strength of a given soil-solid interface.

The majority of the early work in this field has been focused on studying the shear strength of soil-solid interfaces involving freely-draining coarse-grained soils. Often, various modified versions of the conventional direct shear apparatus have been used to determine the interface shear strength between sand and various construction materials. Limitations of the conventional direct shear device have led to the development of various other devices such as the direct simple shear device, the dual interface shear device, and the ring shear device (Kishida and Uesugi, 1987; Paikowsky et al., 1995; Rinne, 1989; Uesugi and Kishida, 1986b; Uesugi et al., 1988; Yoshimi and Kishida, 1981). While these devices provide useful data for many geotechnical engineering
applications, there are considerable limitations that arise when testing interfaces under stress conditions commonly encountered in offshore environments. Primarily the limitations arise as a result of the need to test interfaces under very low effective normal stresses and under relatively large strains in which case these devices do not produce reliable data. Secondly, most offshore developments have to be undertaken in fine-grained soil environments and the traditional devices have certain limitations with respect to testing for the interface shear strength involving fine-grained soils. Hence, attempts are now being made to develop new methods that can overcome some of the limitations to some extent. The next sub-sections provide an overview of the commonly used tools and methods in this field of study.

### 2.2.1 Direct shear apparatus

The conventional direct shear apparatus is often modified to accommodate a solid surface to induce shear at the soil-solid interface (ASTM-D5321-12, 2012). The lower half of the device that usually holds soil is often used to hold a plate or a block of the solid surface to be tested. The solid plate or block is often fabricated to fit inside the lower half of the device (see Fig. 2.2-a). Alternatively the lower half of the device is enlarged in the direction of shear so that a larger surface area in the direction of shear displacement is obtained (see Fig. 2.2-b).

![Typical configurations of the modified direct shear apparatus used for interface shear testing.](image-url)
2.2. Laboratory Testing of Soil-Solid Interface Shear Strength

Testing for the interface shear strength using the modified direct shear device is identical to the conventional direct shear test. The soil specimen is sheared against the solid surface in a displacement controlled manner under a constant normal stress until the required horizontal displacement is reached or until the limiting horizontal displacement of the device is reached. The shear displacement, vertical displacement of the loading cap, and the shear force are measured continuously during shearing. The interface shear strength is calculated based on the shear strength envelope obtained from a series of tests and assuming a Mohr-Coulomb type failure criterion.

The direct shear apparatus presents two important advantages: (i) wide commercial availability; and (ii) relatively simple test setup and sample preparation procedures. Consequently, it has been the common choice for interface testing in research and practice. Other applications of the direct shear apparatus include testing interfaces such as soil-geomembrane, soil-geotextile, and geomembrane-geotextile. Commercial devices have been developed for larger interface areas of up to 305 by 305 mm\(^2\) (Gilbert et al., 1996; Seed and Boulanger, 1991).

The direct shear apparatus present several limitations however. The maximum relative displacement that can be attained in a conventional device is limited; hence, the determination of the large-displacement interface shear strength becomes difficult. Multiple reversals of shear is often required to obtain large-displacement shear behaviour. However this does not always simulate actual field conditions and may not always be reliable. In addition, end effects, induced by the presence of the rigid walls of the soil container, may introduce errors in the test results (OSullivan et al., 2004; Potts et al., 1987). The extreme non-uniformity of strains across the soil specimen will result in undervalued peak shear strengths.

Although not exhaustive, a summary of selected interface shear studies performed using various types of modified direct shear apparatus is presented in Table 2.2. The studies presented in the table have been selected based on the relevance to the interface shear testing that would be common in offshore pipeline applications. Primarily the focus is on interfaces involving coarse-grained or fine-grained soils and steel or concrete surfaces.
Table 2.2: Summary of previous research on soil-solid interfaces conducted using the direct shear apparatus

<table>
<thead>
<tr>
<th>Reference</th>
<th>Interface</th>
<th>Outline of tests conducted</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Potyondy, 1961)</td>
<td>Sand/Steel, Sand/Concrete</td>
<td>60 mm × 60 mm and 90 mm × 90 mm shear area used. Monotonic shear under constant normal load. Investigated the peak interface shear strength only. Smooth and rough surfaces have been tested.</td>
</tr>
<tr>
<td>(Butterfield and An-drawes, 1972)</td>
<td>Sand/Steel</td>
<td>Leighton Buzzard - B.S.S. 14-36 sand was tested against polished mild-steel.</td>
</tr>
<tr>
<td>(Acar et al., 1982)</td>
<td>Sand/Steel, Sand/Concrete</td>
<td>Monotonic shear under constant normal load.</td>
</tr>
<tr>
<td>(Uesugi and Kishida, 1986a)</td>
<td>Sands/Steel</td>
<td>400 mm × 100 mm shear area used. Fujigawa sand and Seto sand tested against low-carbon steel of various surface roughness. Monotonic shear under constant normal load. Investigated the peak interface shear strength and factors of influence.</td>
</tr>
<tr>
<td>(Tejchman and Wu, 2005)</td>
<td>Sand/Steel</td>
<td>Karlsruhe sand was tested against steel of various surface roughness. Monotonic shear under constant normal load.</td>
</tr>
<tr>
<td>(Frost et al., 2002)</td>
<td>Sand/Steel</td>
<td>Modified direct shear apparatus having shear area of 12.7 cm² used. Ottawa 20/30 sand and Valdosta blasting sand was tested against hardened steel and concrete surfaces. Monotonic shear under constant normal load.</td>
</tr>
<tr>
<td>(Dietz and Lings, 2006)</td>
<td>Sand/Steel</td>
<td>Monotonic shear under constant normal load. A winged direct shear apparatus having a 100 mm × 100 mm shear area was used. Coarse, medium and fine sands was tested against mild steel surfaces of various roughness.</td>
</tr>
<tr>
<td>(Tsubakihara and Kishida, 1993)</td>
<td>Clay/Steel</td>
<td>100 mm × 100 mm shear area used. Monotonic shear under constant normal load or constant volume. Kawasaki marine clay tested against low-carbon steel surfaces of various roughness.</td>
</tr>
<tr>
<td>(Tanaka, 2003)</td>
<td>Silt/Steel, Clay/Steel</td>
<td>Gap-controlled direct shear apparatus used. Monotonic shear under constant volume. Silt and Kaolinite clay was tested against steel surfaces of various roughness. A range of shear displacement rates have been used.</td>
</tr>
</tbody>
</table>
### 2.2.2 Direct simple shear apparatus

The use of a simple shear type apparatus to study soil-solid interface shear strength was first presented by (Uesugi and Kishida 1986b) where they developed a new rectangular simple shear device made from a container consisting of stacked aluminium plates with internal openings of 400 by 100 mm, which is the contact area between the soil specimen (29 mm high) and the steel specimen underneath. A constant normal load is applied to the soil, and shearing is imparted by applying a tangential load to the steel specimen. With this apparatus, the sliding displacement at the soil-solid contact surface as well as the displacement due to the shear deformation of the soil mass can be obtained independently. Since the solid surface is longer in the direction of shear, the soil is always in contact with the solid surface during shear. This ensures that the apparent contact area at the soil-solid interface remains constant during shear. Each aluminium ring is coated with a lubricant to minimize inter-ring friction and to allow the rings to deform together with the deformation of the soil mass. A summary of selected interface shear studies performed using various types of modified direct simple shear apparatus is presented in Table 2.3. The capability of measuring the total interface shear displacement and the distortion of the soil mass separately, as shown in Fig. 2.3, is one of the main advantages of direct simple shear device.

Figure 2.3: Typical configuration of the simple shear type apparatus used for soil-solid interface shear testing.
### Table 2.3: Summary of previous research on soil-solid interfaces conducted using the direct simple shear apparatus

<table>
<thead>
<tr>
<th>Reference</th>
<th>Interface</th>
<th>Outline of tests conducted</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Uesugi and Kishida, 1986b)</td>
<td>Sand/Steel</td>
<td>400 mm × 100 mm shear area used. Monotonic shear under constant normal load. Toyoura sand, Fukushima sand, Fujigawa sand, Glass beads were tested against low-carbon steel surfaces of various roughness.</td>
</tr>
<tr>
<td>(Uesugi et al., 1988)</td>
<td>Sand/Steel</td>
<td>400 mm × 100 mm shear area used. Monotonic shear under constant normal load. Seto sand was tested against low-carbon steel surfaces of various roughness.</td>
</tr>
<tr>
<td>(Uesugi et al., 1989)</td>
<td>Sand/Steel</td>
<td>100 mm × 40 mm shear area used. Monotonic or repeated shear under constant normal load. Toyoura sand, Seto sand, Fujigawa sand was tested against low-carbon steel surfaces of various roughness.</td>
</tr>
<tr>
<td>(Evgin and Fakharian, 1997)</td>
<td>Sands/Steel</td>
<td>100 mm × 100 mm shear area used. Monotonic shear under constant normal stress or constant normal stiffness. Dense sand was tested against a rough steel surface. Investigated the peak and large displacement interface shear strength.</td>
</tr>
<tr>
<td>(Fakharian and Evgin, 1997)</td>
<td>Sand/Steel</td>
<td>100 mm × 100 mm shear area used. Cyclic shear under constant normal stress or constant normal stiffness. Silica sand was tested against a rough steel surface.</td>
</tr>
<tr>
<td>(Tsubakihara et al., 1993)</td>
<td>Clay/Steel</td>
<td>60 mm diameter shear area used. Monotonic shear under constant normal load. Kawasaki marine clay and four types of artificially mixed cohesive soils were tested against low-carbon steel surfaces of various roughness.</td>
</tr>
<tr>
<td>(Tsubakihara and Kishida, 1993)</td>
<td>Clay/Steel</td>
<td>60 mm diameter shear area used. Monotonic shear under constant normal load or constant volume. Kawasaki marine clay tested against low-carbon steel surfaces of various roughness.</td>
</tr>
<tr>
<td>(Oumarou and Evgin, 2005)</td>
<td>Sand/Steel</td>
<td>100 mm × 100 mm shear area used. Cyclic shear under constant normal stress. Quartz sand was tested against a rough steel surface.</td>
</tr>
</tbody>
</table>
2.2. Laboratory Testing of Soil-Solid Interface Shear Strength

(Uesugi and Kishida, 1986b) have shown that the horizontal deformation due to distortion of the soil mass is a significant component of the total displacement measured in the direct simple shear device. As for limitations, these devices inevitably carry similar drawbacks as the direct shear device due to non-uniform distribution of stresses at the interface (Kishida and Uesugi, 1987). Also the preparation of the soil specimen is relatively complicated and the device provides limited maximum total displacement, which does not exceed 25.4 mm.

2.2.3 Ring shear apparatus

The conventional ring-shear apparatus has also been used to study soil-solid interfaces. The ring shear apparatus provides a number of advantages over the direct shear and direct simple shear type devices. Namely, (i) the device allows for unlimited circumferential shear displacement at the interface which is preferable in offshore pipeline applications; (ii) the device allows for shearing along the same interface throughout the test; (iii) there is no eccentric loading during shear and stress non-uniformities along the contact surfaces and the soil specimen are reduced to a certain extent. Table 2.4 provides a list of experimental investigations on soil-solid interfaces conducted using ring shear devices relevant to the work covered in this thesis. The ring-shear apparatus has a few disadvantages: (i) relatively complicated in terms of sample preparation and test procedure; (ii) possibility of having a displacement gradient across the interface, and as a result, possible variation of shear strain within the soil specimen; (iii) there is possibility for the test surface to change in texture in large-displacement tests, because of shearing of the soil specimen over the same test surface repeatedly.
Table 2.4: Summary of previous research on soil-solid interfaces conducted using the ring-shear apparatus

<table>
<thead>
<tr>
<th>Reference</th>
<th>Interface</th>
<th>Outline of tests conducted</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Yoshimi and Kishida, 1981)</td>
<td>Sand/Steel</td>
<td>400 mm × 100 mm shear area used. Monotonic shear under constant normal load. Toyoura sand, Tonegawa sand, Niigata sand were tested against low-carbon steel surfaces of various roughness.</td>
</tr>
<tr>
<td>(Ho et al., 2011)</td>
<td>Sand/Steel, Silt/Steel</td>
<td>Range of silica sands, and a silica rock flour silt was sheared against roughened steel annular interfaces. Monotonic shear under incremental constant normal loads.</td>
</tr>
<tr>
<td>(Negussey et al., 1989)</td>
<td>HDPE-geomembrane/sand, HDPE-geomembrane/gravel, HDPE-geomembrane/geotextile</td>
<td>Monotonic shear under constant normal load.</td>
</tr>
<tr>
<td>(Effendi, 1995)</td>
<td>VLDPE, PVC, HDPE, Non-woven geotextiles with high-plasticity clay and Ottawa C-109 sand</td>
<td>Monotonic shear under constant normal stresses within the range: 25 kPa to 300 kPa.</td>
</tr>
<tr>
<td>(Rinne, 1989)</td>
<td>Sand/Steel</td>
<td>Monotonic shear under constant normal load. Ottawa sand, Target sand, was tested against steel surfaces of various roughness.</td>
</tr>
</tbody>
</table>

2.2.4 Other devices

The annular shear type apparatus Brumund and Leonards (1973), the dual interface direct shear apparatus Paikowsky et al. (1995), and more recently, the tile table apparatus Najjar et al. (2007); Pedersen et al. (2003) have been used to study the interface shear strength of soil-solid interfaces. The annular shear type device has many problems associated with stress non-uniformity and is not treated as a favourable apparatus for use in the study of soil-solid interface shear strength. The dual interface shear apparatus is very similar to the simple shear type device described earlier but has the unique feature of providing the ability of determining the shear distribution.
2.2. Laboratory Testing of Soil-Solid Interface Shear Strength

along the interface through the use of a friction bar - see (Paikowsky et al., 1995) for details. The tilt table device provides a simple method of testing for the interface shear strength, but it is difficult to control the rate of shear and loading conditions at the interface since the movement is gravity driven. Further disadvantages are: displacements are not controlled and the post-peak response cannot be measured, test pressures are limited by toppling of the surcharge weights, and non-uniform normal stresses develop on the interface as the device is inclined. Advantages include the elimination of internal machine friction, not forcing failure to occur along the interface, and the ability to perform tests under low normal stresses (Pedersen et al., 2003).

All these devices provide the means to measure the friction between a specimen of soil and a solid surface. However, the use of a particular type of device is primarily dictated by whether the results obtained from the device would be useful in predicting the actual behaviour of a soil-structure interaction problem encountered in the field. It is often difficult (if not impossible) to use these conventional devices to capture the soil-solid interface friction angle under very low effective normal stresses. Problems are often attributed to the mechanical friction related errors that are emphasized at low stress levels, limited displacement ranges, and difficulty in measuring excess pore-water pressure during shear. Testing for the interface shear strength under effective normal stresses greater than 50 kPa are quite common on these apparatus. However, it is often found that these methods are not always directly applicable to offshore pipeline design and require modifications in order to be of good use. In order to overcome such difficulties an attempt was made to develop a large-scale direct shear type interface shear test apparatus that would be of use for the offshore pipeline industry.
Chapter 3

The Macro-Scale Interface Direct Shear Device

The primary goal of the Master’s research project covered in this thesis was the design and commissioning of a macro-scale direct interface shear test device for use in offshore geotechnical engineering research. During the two-year duration of the work a major part of the project involved the fabrication and assemblage of the various components of the new device, trial testing, modification of the device according to the requirements identified from the trial tests, and raw material acquisition. Subsequently the final design of the newly built device was taken through a series of performance tests to evaluate and ensure the proper functionality of the new equipment. An abridged version of the endeavour is initially presented for completeness. This is then followed by a detailed description of the final version of the new device and the performance tests conducted during the study.

The main objective was to develop a new test device capable of determining the drained large-displacement interface shear strength characteristics at soil-solid interfaces under the following conditions:

1. Capability of performing interface direct shear tests at effective normal stresses ranging from 3 kPa to 6 kPa.

2. Primarily for use in determining the large-displacement interface shear strength under drained conditions.

3. Capability of testing coarse-grained soils as well as fine-grained soils against various solid surfaces.
3.1. Overview of Test Device

Testing soil-solid interfaces under very low effective normal stresses stems from the need to quantify the shear strength at the pipe-soil interface for off-shore oil and gas pipeline design applications where the effective normal stress at the pipe-soil interface is within the range of 3 kPa to 6 kPa.

The design and fabrication of the new device commenced in January of 2011 and after many trials and modifications was completed in June of 2012. Many theoretical and practical aspects were considered in the design of the device and each and every component of the new device had to be designed and fabricated from “the ground-up”. In brief, these design aspects included the following: principle of operation, the size and footprint of the soil specimen, the materials to be used for fabrication, methods to reduce the friction of the mechanical components, containment of slurry type soil specimens within the device, application of load for realizing shearing at the soil-solid interface, measurement of required parameters, instrumentation and data acquisition. While most aspects were dealt with based on theoretical calculations and based on information found in technical literature, some had to be finalized by trial and error, which necessitated an array of trial experiments to be conducted that led to several modifications of the device. The following sub-sections provide details of the fully operational device.

3.1 Overview of Test Device

The final version of the device presented in this thesis meets the testing criteria listed above and is fully functional. Primarily based on the conventional direct shear test, the new device has a simple principle of operation. A reconstituted soil specimen, that is confined within a mobile aluminium frame resting on top of a rigid test surface of choice, is sheared against this surface under a constant normal effective stress by applying a constant rate of shear displacement to the mobile frame. The shear displacement and the applied shear force on the mobile frame are measured during shearing. The soil specimen is free to contract or expand during shearing and there are several means available to ensure that shearing is performed under drained conditions.
3.1. Overview of Test Device

Volume change behaviour of the soil specimen is not monitored as the primary purpose of this device is to provide the large displacement interface shear strength for a given interface - the device is not suitable as a means for observing small shear strain behaviour. The general details of the macro-scale interface shear test device are shown in Figs. 3.1 and 3.2.

![Diagram of the macro-scale interface shear test device and test setup.](image)

- Test soil specimen (usually 5 cm to 10 cm thick)
- Test base plates coated with pipeline coating material
- Layer of dead load sand reinforced with a Geocell™ for application of surcharge
- Reinforced stiff mobile shear frame with Plexiglas sidewalls
- Non-reinforced layer of dead load sand or water for additional surcharge load when needed

Figure 3.1: Schematic diagram of the macro-scale interface shear test device and test setup.

The basic principle of operation of the device is identical to that of the well-established standard direct shear device (as per ASTM D 3080-04). As shown in Fig. 3.2 the device comprises a 2 m by 4 m stationary flat base mounted on a rigid frame made of galvanized steel. The stationary flat base is used to support two solid base plates that together serve as the solid test surface of known texture. On top of the stationary base is a rigid aluminium mobile frame that has four wheels that are connected to the frame through axles located at the North and South ends.
3.1. Overview of Test Device

Figure 3.2: Photograph of the macro-scale interface shear test device.

The wheels support the mobile frame on two rails that run along the East and West sides of the stationary base. This design allows the mobile frame to move freely with minimal frictional resistance by providing a clearance between the mobile frame and the solid test surface.

The mobile frame has four acrylic poly(methyl-methacrylate) side-walls extending upward from the top. As shown in Fig. 3.1, the mobile frame serves as the containment for the test soil specimen and provides an interface shear area of 1.72 m by 1.75 m (approximately 3 m²) in plan. Thin, flexible rubber wipers are mounted on the interior of the four aluminium side-walls of the mobile frame at the base so that the soil specimen does not escape through the clearance between the mobile frame and the test surface. Also, when fully saturated soil specimens are tested, the wipers ensure that water does not drain from the sides of the mobile frame at the soil-
solid interface. The device can be used to test coarse-grained as well as fine-grained materials under drained conditions. The soil specimen can be fully saturated in cases where fine-grained soils are used. Pressure transducers that are attached flush to the test surface are used to monitor pore-water pressure generation during shearing. These measurements provide the means to calculate the effective normal stress at the interface. The basic steps involved in the preparation of the test soil specimens of coarse-grained and fine-grained soils are described in a separate section. The details of the testing mechanism including details related to the application of vertical stress to the test soil (i.e., the preparation requirements for soil surcharge) are given in the following section.

3.2 Description of Main Components of Test Device

A detailed description of the test device, especially with reference to its key components is provided in the following sub-sections.

3.2.1 Support frame and stationary base

A rigid frame structure constructed of galvanized steel members (see Fig. 3.3) serves as the reaction frame for the test device and supports the stationary base made of two 0.5 inch thick plywood sheets stacked on top of each other. This flat base measures approximately 2.0 m by 4.0 m in plan and is located approximately 1 m above the ground surface. It is lined with two rigid plates (placed side by side) made of or coated or lined with the test surface of required texture. For example, if the interest is to obtain the interface friction between a given soil and an epoxy coated steel surface, the epoxy coating material(s) is pre-applied to two rigid steel plates, which are then mounted securely on to the stationary base. Each test plate measures approximately 1.9 m by 1.5 m in plan.

The test surface can be of single-plate construction, but this can make handling
3.2. Description of Main Components of Test Device

Figure 3.3: Plan and end elevation of the stationary base.
3.2. Description of Main Components of Test Device

and interchanging of test plates difficult. The North side of the support frame houses the stepper motor and the worm-and-gear mechanism. The worm-and-gear mechanism is used to transform the rotary motion of the stepper motor axle into the desired linear motion of the mobile frame. The East and West sides of the support frame are equipped with galvanized steel rails that support the wheels of the mobile frame. The support frame is secured to the ground at the bases of the columns.

![Diagram of test device components](image)

Figure 3.4: Photograph of test plates attached to the stationary base showing spacial distribution of pore-water pressure sensor apertures.

The 1 m clearance between the ground and the stationary base allows access to the underside of the base. Circular openings are provided in the stationary base allowing direct access to the underside of the test plates. The test plates have circular apertures that align immediately above the openings of the base, in turn, allowing
the installation of pressure transducers, to measure the pore water pressures at the soil-solid interface, from the underside of the stationary base. These apertures are approximately 6 mm in diameter and they allow having the surface of the pore water pressure transducers to stay flush with the top of the test surface. The very first operational version of the new device was equipped with only three pore-water pressure transducers in order to investigate whether the use of pore-water pressure transducers would provide valuable information on pore-water pressure variation during testing of fully-saturated fine-grained soils. This indeed proved to be the case and afterwards, seven pore-water pressure transducers were installed in the final version of the device. The spacial distribution of the pore-water pressure transducer apertures are shown in Figs. 3.3-a, and 3.4. This layout of seven pressure transducers ensures that an optimal planar area of the soil specimen is monitored for excess pore-water pressure during testing.

3.2.2 Mobile frame and actuator

The mobile frame is of rigid aluminium construction and is supported on axles with the wheels moving on rails mounted on the East and West periphery of the top of the support frame. A rigid transparent poly(methyl-methacrylate) enclosure, which provides containment for the test soil and at the same time acts as an observation screen, is mounted onto the top of the mobile aluminium frame. Fig. 3.5 shows the plan and end elevation views of the mobile frame. As shown in Fig. 3.5-a, the mobile frame provides a 1.80 m by 1.78 m area in plan that is open to the test surface at the base. Once supported on the rails of the support frame, the clearance between the bottom of the mobile frame and top of the test surface is approximately 5 mm as shown in the enlarged portion of the mobile frame in Fig. 3.5-b. The bottom wheels rest on the bottom rail of the support frame and are used to support the weight of the mobile frame and to guide the displacement in the horizontal direction. The top wheels that rest against the top rails of the support frame are used to restrict any vertical movement of the mobile frame during shearing.

The 5 mm clearance between the mobile frame and the test surface is closed by
3.2. Description of Main Components of Test Device

Figure 3.5: Plan view and end elevation of the mobile frame.

attaching a flexible rubber wiper to the inner wall of the mobile frame such that the wiper is in contact with the test surface as shown in Fig. 3.6. The use of a thin, flexible rubber wiper ensures that there is minimal frictional resistance to the horizontal displacement of the mobile frame but also serves the purpose of effectively retaining the soil specimen within the footprint of the mobile frame and at the same time seals drainage from the bottom periphery of the specimen.

The selection of the rubber wiper was made by performing a number of trail runs without a soil specimen in place to observe the frictional resistance that the wiper rubbing against the test surface would produce. It was found that a fairly thin and extremely flexible wiper made of rubber produce the minimum frictional resistance. When all four wipers are attached to the four sides of the mobile frame, the total frictional force measured was approximately 0.1 kN. This is roughly about 1% of the interface shear resistance expected and is acceptable. Nevertheless, before each interface shear test, the device is tested for the frictional resistance due to the wiper rubbing against the test surface and this data is used to correct actual interface shear test data. Fig. 3.7 shows a photograph of the mobile frame with the wipers attached to the side walls. Note that the addition of the wipers slightly reduces the area that
3.2. Description of Main Components of Test Device

is exposed to the test surface at the base of the mobile frame.

![Diagram of test device components](image)

Figure 3.6: Details of rubber wiper used to seal the gap between the mobile frame and test surface. (Cross-section of the North end of the mobile frame shown).

The mobile frame is displaced precisely using a worm and gear arrangement connected to a commercially available stepper motor (Input: DC 2.5 A, speed: 1500 rpm, 1.8 degree step motor bipolar series, power and voltage: 316 W, 170 V). As shown in Figs. 3.2 and 3.8, a 2.0-m long threaded rod serves as the worm in the mechanism and is used to transfer the pulling force to the mobile frame (pulling force is applied towards the North direction). The stepper motor controller is interfaced with a software program through which the direction of rotation and speed of the motor is controlled. The device is capable of a total shear displacement length of 1.2 m at displacement rates from as low as 0.0001 mm/s to a maximum of 1 mm/s. The displacement rate can be defined in the control software and is kept constant over the duration of a given interface shear test. The low displacement rates attainable in this device can be used to test fully saturated fine-grained soils and ensure that there is
minimal excess pore pressure generation at the interface during shearing.

### 3.2.3 Sensors and data acquisition

**Measurement of shear force and shear displacement**

A load cell of 44 kN (10,000 lb) capacity and a string potentiometer of 2.0 m measurement range attached to the mobile frame are used to measure the load and displacement induced during interface shear tests. As shown in Fig. 3.9, the threaded steel rod that transfers the pulling force to the mobile frame is connected to the load cell across a double-pin type connection so that no bending or torsional moments and only the horizontal pulling force is transferred to the mobile frame during the pulling process.
3.2. Description of Main Components of Test Device

The load cell is attached to the mobile frame through a stiffener I-beam that is fastened to the North peripheral side of the mobile frame (see Fig. 3.10). The load cell is positioned in such a way that the axis of the load cell passes through the centroid of the aluminium mobile frame. With the test plates in place, the axis of the load cell lies approximately 75 mm above the test surface. The two-pin connection cannot be used to transmit compressive forces and is only usable in tension. When a tensile load is applied across the two-pin link, the axis of pull coincides with the axis of the load cell. This ensures that no bending moments are transferred to the load cell during testing. When no tensile load is applied across the pin connection, the tie rod remains loose and the threaded rod sags at the pin connection. Prior to starting any interface shear test, a minute tensile load can be applied to the mobile frame to engage the pin connection. The voltage corresponding to this load can be
3.2. Description of Main Components of Test Device

Figure 3.9: Photograph illustrating the hinged connection between the load cell and the threaded rod transferring the tensile force to the mobile frame. (Note that the hinge shown, is in a state with no load applied hence the slack at the hinge in the photograph).

recorded as the initial load cell reading prior to start of test. This initial voltage reading of the load cell can be used to get a rough idea as to whether the load cell is functioning properly. Nevertheless, the load cell has been calibrated many times during the development of the device which was a necessary check carried out after each modification that could potentially affect the readings of the load cell. The load cell response within range of loads expected is linear showing a calibration factor of 11.1 kN/V (per 10 V excitation) and a resolution of 0.022 kN (i.e. 0.002 V x 11.1 kN/V).
3.2. Description of Main Components of Test Device

**Figure 3.10:** Details of pin connection between the load cell and the threaded rod.

**Measurement of pore-water pressure**

The device is equipped with pressure transducers (Model: SPT4V0015PA4W02, Type: Gauge, Range: 0 to 15 psi, resolution: 0.006 kPa, Manufacturer: Honeywell Sensing and Control, USA) that are used to measure pore-water pressure generation at the interface (see Fig. 3.11). The pressure transducers show a linear response within the range of pore-water pressure expected during testing and a calibration factor of 1.10 kPa/V (per 12 V excitation). The pressure transducers are mounted to the bottom of the test plates through a stainless steel adapter such that the opening of the adapter is flush with the top surface of the test plates (see Fig. 3.12). As described in section 3.2.1 the spatial distribution of the pressure sensors on the test surface has been established based on ensuring optimum coverage of the soil specimen during shearing (see Figs. 3.3 and 3.4 for details).

The pressure transducers and the stainless steel adapters should be well cleaned and fully saturated with either de-aired, incompressible liquid medium such as glycerine or distilled water before placement of the soil specimen into the mobile frame. Preparation of the pressure transducer configuration is done by first attaching the stainless steel adapters to the test plates from underneath the stationary base, and
3.2. Description of Main Components of Test Device

then attaching the pressure transducers to the adapters. Each adapter is cleaned thoroughly and attached to the test plate prior to attaching the pressure transducers. Then each pressure transducer is saturated with the appropriate liquid by using a syringe with a flexible rubber tube fixed to the needle at the end. This is done by first inserting the tube into the cavity of the pressure transducer all the way to the base and then filling the cavity with the liquid from the base upward. Special care is taken to ensure that no air bubble are entrapped inside the cavity during this procedure. Once the cavity is completely full a meniscus of the liquid covers the opening as shown in Fig 3.13-a,b. The saturated pressure transducer is then carefully inserted into the adapter and screwed in tight. Using the same syringe, and with the help of the rubber tube, the pressure transducer and adapter cavity is carefully filled with the liquid starting from the base of the transducer cavity all the way to the opening of the adapter at the test surface (see Fig. 3.13b,c). At the end of this operation a meniscus is left on top of the adapter opening. This opening is then covered with a porous stone that fits tight inside the opening. The porous stone should also be
saturated with the appropriate liquid before it is used to cover the opening of the adapter. This is done by keeping it submerged in glycerine while inside an ultrasonic bath or by keeping it submerged in water under a vacuum for several days.

The data acquisition system can be used to monitor the pressure transducer readings during the saturation process. Keeping a record of the pressure transducer voltage at the end of saturation helps to check whether the saturation has been done properly. There are occasions when the pressure transducer readings post-saturation appear doubtful. In these situations, a check is made to ensure that all transducers are functioning properly. This is done by placing an acrylic hollow cylinder on top of the test surface such that one opening of the cylinder sits right above the opening of the adapter. The opening of the cylinder is secured to the test surface by first applying a layer of silicone grease onto the edge of the opening of the cylinder and then pressing it tight onto the test surface. This produces a water-tight connection. The cylinder is then carefully filled with water to a confidently measurable height. The water head produced thus can be calculated and the pressure transducer reading should show the corresponding head, in turn, allowing the checking of the accuracy.
3.2. Description of Main Components of Test Device

(a)(b)(c)
Syringe
Flexible rubber tube
De-aired glycerine or, de-aired distilled water

Figure 3.13: Details of pressure transducer saturation procedure. (a) Saturation of pressure transducer cavity. (b) Attachment of saturated transducer to stainless steel adapter and saturation of adapter. (c) Placement of porous stone.

Data acquisition system

The load cell and the pore-water pressure transducer signals pass through a signal conditioner, and then through an 16-bit analog-to-digital (A/D) converter after which the signals are saved and processed on a personal computer. The signal conditioner is a model that was designed and built at UBC. It has a total of 16 channels out of which eight are used for sensors of the device. The A/D converter is a National Instruments USB-6221 type that has 16 input/output channels. The signal conditioner is used to filter out noise in the system and also to amplify the signals and the A/D converter converts the analog signals to digital form. The digital data is sent to a personal
computer via a universal serial bus (USB) connection. The data is processed using the National Instruments Signal Express software (version: 2.5.1) with NI-DAQ driver version 9.2.0. The data acquisition system is used to sample data at a rate of 1 sample/second which is sufficient for the type of testing that is intended to be carried out on the device. The user has control over the sampling rate and this rate can be increased up to 100 samples/second without losing system stability.

3.2.4 Application of effective normal stress

It is important that the vertical effective stress that is applied on the soil specimen is uniform across the plan area of the specimen, that it is kept constant during the test, and that it does not severely affect drainage conditions at the drainage boundary. In this respect, the application of a very low effective normal stress (3 kPa to 6 kPa) on a relatively large planar area (3 m$^2$) was found to be a challenging task. Three different options were considered to achieve this: (1) using an air bladder on top of the soil specimen, (2) placement of rigid aluminium tanks on top of the soil specimen and filling the tanks with water, (3) use of self-weight of a granular material placed on top of the soil specimen as a surcharge load. The use of an air bladder was found to be the most effective option of the three but this was not found to be a viable option given that the development of the device was to be completed in two years. The use of aluminium tanks was experimented with and was found to be not effective in applying a uniform vertical stress on the 3 m$^2$ area. The use of a granular material as a surcharge was experimented and after several major modifications, this approach was found to produce very good results. Details of these various experiments are not presented in the text for brevity.

Based on the above, the desired effective normal stress at the test-soil/baseplate interface is generated using the self-weight of the test soil specimen supplemented by the self-weight of a surcharge soil layer having a constant vertical thickness and is reinforced by a cellular grid. Uniformly graded Fraser River sand was used as the surcharge soil in the tests that are documented in this paper. Placing the sand is done manually and care is taken to ensure that the sand is placed uniformly on top
of the test soil specimen inside the mobile frame until the required thickness of the sand layer is achieved. Usually a 15 cm thick sand layer in combination with a 10 cm thick test soil specimen provides a vertical effective stress of 3 kPa. In certain cases an additional surcharge load is applied on top of the already placed surcharge sand layer to achieve vertical effective stresses greater than 3 kPa (see Fig. 3.14). This is achieved by first covering the surcharge sand and the transparent poly(methyl methacrylate) walls with two or three sheets of 6-mil polyethylene, thus creating an impermeable water barrier on top of the surcharge sand, and then filling the created space with water up to the required height. Note that in order to minimize the side-wall friction, the side-walls of the mobile frame are lined with woven geotextile sheets and two layers of polyethylene sheeting (Fig. 3.14) prior to placing the test soil specimen as well as the surcharge sand layers. Tognon et al. (1999) have observed that the use of double-polyethylene lined walls is an effective way of reducing side wall friction even in the case of chamber test walls that are conducted at significantly high stress levels than those considered herein. A sheet of woven or non-woven geotextile is used between the test soil layer and the surcharge sand layer as a filter, thus preventing contamination of the lower test soil layer from the surcharge sand while still allowing free drainage of water across the layers. A commercially available non-woven geotextile having an apparent opening size of 0.212 mm (ASTM-D4751) and a permittivity of 1.3 s\(^{-1}\) (ASTM-D4491) was found to be appropriate as the separation layer when fine-grained soils are tested; the non-woven geotextile served effectively as a drainage boundary without allowing the loss of material. A commercially available, monofilament polypropylene yarn based woven geotextile having an apparent opening size of 0.212 mm (ASTM-D4751) and a permittivity of 0.28 s\(^{-1}\) was found to serve the purpose when the test soil was coarse-grained. This configuration of the specimen and surcharge-loading layers results in a single-drainage path at the top surface of the specimen.
3.2. Description of Main Components of Test Device

Figure 3.14: Schematic cross-section showing test specimen. Surcharge sand having a constant thickness placed over the test soil/base-plate interface. (Not to scale).
3.3 Details of Surcharge Loading

As mentioned in the previous section, the desired effective normal stress at the test-soil/test-plate interface is obtained using the mass of surcharge sand sometimes combined with a mass of external water surcharge. In order to apply a uniform normal stress on the test soil specimen, and to ensure that the normal stress remains constant throughout the duration of the test, it was decided that the surcharge sand layer be reinforced with a cellular geosynthetic grid so that the surcharge sand stays intact as a block having a uniform height but is free to undergo vertical deformation if necessary. This is accomplished with the use of a high-density polyethylene (HDPE) based cellular confinement grid (i.e., geo-cell grid). Essentially, once the test soil specimen is prepared inside the mobile frame (the test soil specimen is usually 10 cm thick, details of specimen preparation are provided in the later in the text) and is lined with the geosynthetic filter layer, the geo-cell grid (having expanded plan dimensions of 1.70 m by 1.72 m and a height of 0.15 m) is carefully placed on top as shown in Fig. 3.15.

The geo-cell is placed so that none of the edges of the geo-cell would touch the side-walls of the mobile frame. The void space within the geo-cell is then filled with the surcharge sand and subsequently levelled (Fig. 3.16). The sand placement is done using a small capacity scoop (100 cm$^3$) in small lifts so that there is minimal disturbance to the test soil specimen below the geotextile filter layer. The placement is made such that the surcharge sand level would rise gradually and evenly over the test footprint, so that there would be no opportunity for local shearing of the underlying test soil, due to unevenly distributed surcharge load. Empty containers of known volume are embedded in the sand layer during filling so that the density of the sand surcharge could be determined (before and after each shear test) for the calculation of the effective vertical stress. Once the sand placement is completed, the thickness of the backfill sand layer is measured and recorded.

In cases where fully saturated fine-grained soils are tested, the increase in the pore-water pressure at the test plate level due to the gradual and uniform filling of
3.3. Details of Surcharge Loading

Figure 3.15: Placement of geo-cell grid on top of the soil specimen in preparation of placement of surcharge load sand layer. (a) Soil specimen lined with the geotextile filter layer. (b) Geo-cell confinement grid placed on top of the test specimen and ready to be filled with sand.
3.3. Details of Surcharge Loading

Figure 3.16: Surcharge load sand layer reinforced with geo-cell grid prepared on top of the soil specimen.

the surcharge sand layer is monitored continuously. The total surcharge load is determined based on the final thickness and the density of the placed sand layer and the weight of the geo-cell grid together with the non-woven geotextile separation layer. The pore-water pressure at the end of application of the surcharge load is then compared with the actual applied surcharge load to check whether the measured increase in pore-water pressure at the base of the specimen is at least 95% of the applied surcharge load to ensure that the fine-grained test soil specimen is fully saturated. Based on the density of surcharge sand measured before and after the test (usually 16 to 17 kN/m$^3$) and taking the weight of the geo-cell (7.02 kg) and the geotextile sheet (1.55 kg) into account, the total normal stress on the test-soil/base-plate interface
3.4. Details of Specimen Preparation

can be readily calculated as follows:

\[
\sigma_n = h_{ts} \gamma_{ts} + h_{rs} \gamma_{rs} + \frac{w_{gc} + w_{gt} + w_{ps}}{A_i} + h_w \gamma_w
\]  

(3.1)

where, \( \sigma_n \) = total normal stress at the test-soil/test-plate interface (kPa), \( h_{ts} \) =average thickness of test soil layer (m), \( \gamma_{ts} \) =average unit weight of test soil layer (kN/m\(^3\)), \( h_{rs} \) =average thickness of surcharge load soil layer (m), \( \gamma_{rs} \) =average unit weight of surcharge load soil layer (kN/m\(^3\)), \( w_{gc} \) =weight of geo-cell grid (kN), \( w_{gt} \) =weight of geotextile filter layer (kN), \( w_{ps} \) =weight of impermeable polyethyylene sheeting (kN), \( A_i \) =plan area of test-soil/test-plate interface (m\(^2\)), \( \gamma_w \) =unit weight of surcharge load water at room temperature (kN/m\(^3\)), and \( h_w \) =thickness of water layer used to apply the additional surcharge load when needed (m). It was noted that a sand backfill of approximately 15 cm thick combined with a test soil layer of approximately 5 cm to 10 cm thick could provide an effective normal stress of approximately 3 kPa to 4 kPa at the interface.

3.4 Details of Specimen Preparation

The basic procedure followed in the preparation of a soil specimen for interface shear testing on the new device is described below. As indicated earlier, the process herein involves the placement of the test-soil material within the footprint of the mobile frame having plan dimensions of 1.72 m by 1.75 m. As a common step, applicable to the preparation of both, the coarse-grained and fine-grained soil specimens, the base plates and the mobile frame are first prepared for receiving the test soil specimen. This involves:

1. Thorough cleaning of the test surface (base plates) and the interior side-walls of the mobile frame.

2. Lining the interior side-walls of the mobile frame with a single layer of woven geotextile and a dual layer of polyethylene sheeting. This is used to reduce the
side wall friction acting on the specimen and the surcharge load sand layers (Tognon et al., 1999).

3. Testing the device for mechanical device friction (friction associated with the mechanical components of the device when not soil specimen is present inside the mobile frame.

4. Attachment of pore water pressure sensors to the test plates and saturating the sensors with de-aired, distilled water or de-aired glycerin (this step is necessary only when a fully saturated soil specimen is to be used in the test).

In order to reduce the side-wall friction imparted on the soil specimen and the surcharge load sand layer the side-walls of the mobile frame are are lined with two layers of 6-mil polyethylene. Also the interior of the aluminium mobile frame is lined with a layer of monofilament polypropylene yarn based woven geotextile having an apparent opening size of 0.212 mm (ASTM-D4751) and a permittivity of 0.28 s\(^{-1}\) (see Fig. 3.17). The geotextile side-skirts provide a filter layer that prevents the soil specimen from escaping the confinement of the mobile frame during testing. Once the side-wall have been prepared in this manner the mobile frame is ready to accept the soil specimen. The next two sections provide details of soil specimen preparation.

### 3.4.1 Preparation of coarse-grained soil specimens

Preparation of the soil specimen involves placing the test soil material inside the 1.72 m by 1.75 m footprint of the mobile frame as uniformly as possible until a desired test soil layer thickness is achieved. Prior to specimen preparation the test soil is dried either using an oven or by natural evaporation under room temperature. Having an dry soil mass aids in producing a uniform distribution of the test soil material inside the mobile frame during placement. The test soil is then air-pluviated into the mobile frame from an approximate height of 1 m from the base plate level in many batches using a container of approximately 1 litre capacity. The soil specimen is spread over the footprint of the mobile frame and the thickness of the specimen is calculated from
3.4. Details of Specimen Preparation

Figure 3.17: Preparation of side-walls of the mobile frame for receiving soil specimen. (a) Non-woven geotextile side-skirts attached to interior of the aluminium mobile frame. (b) Side-walls of the mobile frame lined with two layers of 6-mil polyethylene sheeting.

level measurements taken from the top of the mobile frame to the surface of the soil specimen. Fig. 3.18 shows a soil specimen that has been prepared in this manner.

Several small containers are embedded in the test soil during soil placement. These containers are retrieved upon completion of the specimen preparation and are used to calculate the as-placed density of the soil specimen. Usually a 5 cm to 10 cm thick layer of soil is produced to form the test soil specimen. Several containers that are embedded in the specimen during soil placement are left inside the specimen for retrieval at the end of testing. These containers are used to determine the post-test unit weight of the soil specimen.
3.4.2 Preparation of fine-grained soil specimens

Testing of fine-grained soils are carried out under fully saturated, fully drained conditions. The calculation of the effective normal stress at the test-soil/test-surface interface is dependent upon on the accurate measurement of pore-water pressure at the interface during testing. Hence special care is taken to ensure the fine-grained soil specimen is fully saturated. Essentially a slurry deposition method is used to prepare the fine-grained soil specimen. The method involves (a) preparation of a slurry of the fine-grained soil by mixing the soil with water until a moisture content of approximately 150% is reached, and (b) allowing the slurry to flow into the mobile frame under gravity to fill the 1.72 m by 1.75 m footprint until the desired thickness of test soil specimen is achieved. Special care is taken to minimize any turbulence in the flow that could result in the entrapment of air pockets within the slurry.
3.4. Details of Specimen Preparation

The following steps are followed as a part of this slurry-making process:

1. The moisture content of the as-supplied fine-grained soil is first determined.

2. Based on the as-supplied moisture content and the index properties of the soil, the mass ratio of water to fine-grained soil required to produce a mixture having a moisture content well above the liquid limit of the fine-grained soil is determined.

3. In a plastic mixing container (see Fig. 3.19) of approximately 124 litre capacity, the fine-grained soil is mixed with water at the previously calculated mixing ratio. An electric mixer is used for mixing the material until a homogeneous slurry is produced. Special care is taken to ensure that a fully saturated slurry is achieved. The plastic mixing container is equipped with a (50-mm diameter) valve at the base protruding out from the side wall to serve as the exit port for releasing the mixed material under gravity as needed. The slurry is let to flow through a hose into the mobile frame under gravity (see Fig. 3.20).

4. Samples of the prepared slurry are obtained to keep a record of the moisture content of the prepared slurry.

Placement of the slurry in the device starts with opening the valve to allow the slurry to flow into the mobile frame under gravity through a 50-mm diameter hose. The valve is located approximately at a height of 1 m from the base plate level when the slurry is released into the mobile frame. The slurry placement is commenced at the center of the mobile frame and the slurry freely spreads across the test surface. Several batches of slurry are required to produce a sufficient thickness of slurry inside the mobile frame. Usually a 5 cm to 15 cm thick layer of slurry is required to produce a fine-grained soil specimen suitable for testing (see Fig. 3.21).

Pore-water pressure at the test surface is continuously measured during placement of slurry. Once placed, level measurements are taken from the top of the mobile frame to the surface of the as-placed slurry layer to determine the thickness of the layer. The slurry is then allowed to settle on its own weight. For non-plastic Fraser river silt
3.4. Details of Specimen Preparation

Figure 3.19: Plastic container of 124 L capacity and the electric mixer used in the preparation of the slurry.

specimens this consolidation process takes around 24 hours to complete. For kaolinitic soils the process can take around three to five days to complete. At this point, more samples of the settled slurry are taken to determine the moisture content, and the thickness of the settled slurry layer is measured. Note that at 150 % moisture content, the slurry takes the form of a liquid which can show low to high viscosity depending on the soil material. It is preferable to prepare a slurry that is very low in viscosity so that (i) potential for the entrapment of air pockets during mixing and placement is reduced, and (ii) to ensure that the deposition (settlement) of the solid material occurs evenly under gravity after the slurry has been placed inside the mobile frame. The settlement process that takes place after placing the slurry inside the mobile frame is analogous to the settlement of fine-grained soil in a hydrometer test. During
the settlement process, the solid material displaces some of the water out of the soil matrix; and at the end, a layer of clear water can be observed above the surface of the soil specimen. By measuring the level height to this water surface from a known datum, the thickness of the layer of water and the corresponding static head can be calculated (see Fig. 3.23). The absolute pore-water pressure recorded from the pressure transducers should be equal to this static head if the pressure transducers are directly exposed to this layer of water at the test surface. Indeed this was proven to be the case and it served as a way to check that the pressure transducers would show the calculated static head prior to application of surcharge load. The surface of this self-weight consolidated slurry layer is then covered with a non-woven geotextile layer in preparation for surcharge loading. The desired surcharge load is applied (as described in the section 3.3) and the fine-grained soil specimen is allowed to consolidate under
3.4. Details of Specimen Preparation

Figure 3.21: Fine-grained soil specimen after placing inside the mobile frame.

This load until the excess pore-water pressure generated during the placement of the surcharge is adequately dissipated. It was possible to use the consolidation curve (pore-water pressure dissipation curve) obtained during this process for estimating the coefficient of consolidation of the test-soil (see Fig. 3.22). The pore-water pressure before placement of the surcharge load (denoted as \( a \) in Fig. 3.22) is equal to the static head of the column of water that is formed during the self-weight settlement of the fine-grained soil slurry.
Given the low hydraulic conductivity of fine-grained soils, the application of the surcharge load is immediately reflected as an increase in the pore-water pressure measured by the pressure transducers at the soil/solid interface. The pore-water pressure rise during the gradual and uniform placement of the surcharge sand layer is
3.5 Selection of Appropriate Shear Displacement Rate

reflected in readings obtained from the pore-water pressure sensors as shown by the region \( b \) in Fig. [3.22]. The sudden *break* in the pore water pressure rise at around 2.5 kPa in region \( b \) of Fig. [3.22] is because the placement of sand was temporarily stopped during the swapping of containers that are used to store the sand material - two containers, each holding approximately 0.4 m\(^3\) of sand are usually required to prepare the surcharge sand layer. Once the sand containers were swapped, the application of surcharge load was continued until the desired amount of surcharge load was reached. The total surcharge load applied is reflected in the pore water pressure readings as shown by point \( c \) in Fig. [3.22]. After applying the surcharge load, the specimen is allowed to consolidate until the excess pore water pressure dissipated. For fine-grained soils the consolidation process can take from 7 to 30 days depending on the material and thickness of the specimen. Finally at the end of consolidation, the thickness of the consolidated soil layer is calculated, at which point the specimen is ready for testing.

3.5 Selection of Appropriate Shear Displacement Rate

The excess pore-water pressure dissipation characteristics obtained from the series of pore-water pressure transducers (e.g., Fig. [3.22]) provide an opportunity to calculate a rate of shear displacement that would promote minimal excess pore-water pressure generation during the shearing stage. The soil specimen is constrained within the boundaries of the four side-walls of the mobile frame on the North, South, East, and West sides, and is also constrained at the bottom by the test-surface. Free drainage of pore-water is only possible across the top of the soil specimen where the water drains into the surcharge sand layer through a geotextile filter. These drainage boundary conditions are similar to that used in the formulation of Terzaghi’s one-dimensional consolidation theory. The problem can be simplified to a soil specimen having no radial drainage and only one drainage boundary in the vertical direction. Using the
3.5. Selection of Appropriate Shear Displacement Rate

concepts of one-dimensional consolidation with single drainage on the excess pore water pressure versus time characteristics the coefficient of consolidation $c_v$ of the fine-grained soil specimen can be estimated reasonably well (see Fig. 3.24).

Figure 3.24: Pore-water pressure dissipation profile obtained during consolidation of fine-grained soil specimen used to calculate the coefficient of one-dimensional consolidation of the soil specimen.

Determination of the appropriate rate of displacement requires an estimate of the time required for pore pressure dissipation and amount of deformation required to reach failure. These two factors depend on the type of material and the stress history. The theoretical equation proposed by Gibson and Henkel (1954), as given by Eq. 3.2, together with guidelines specified in ASTM-D3080-98 (1998) can then be used to calculate the time to failure, $t_f$, that would correspond to a given degree of pore pressure dissipation at failure.

$$t_f = \frac{H^2}{\eta c_v (1 - U_f)}$$ (3.2)

where, $H$ = thickness of test-soil specimen = length drainage path,
$c_v$ = coefficient of one-dimensional consolidation of the test-soil specimen,
$U_f$ = desired degree of pore-water pressure dissipation at failure,
$\eta$ = numerical factor corresponding to the extent and location of the drainage boundaries (see highlighted values given in Fig. 3.24).
A value of $\eta$ equal to 2 is recommended by (Gibson and Henkel, 1954) to represent the prevalent drainage conditions in the direct shear test with 1D drainage. The time to failure is assumed to be the time required for the mobile frame to achieve a shear displacement of 300 mm; it is considered that this magnitude of displacement would be sufficient to mobilize the large-displacement interface friction angle. Substituting this rationalized value in Eq. 3.2, the value of $t_f$ corresponding to a pore water pressure equalization of 85% can be calculated. This, in turn, can be used to calculate the required displacement rate by dividing the displacement of 300 mm by $t_f$. Considering the extensive time duration required for the testing of fine-grained soils and that the excess pore-water pressures in the immediate vicinity of the shear zone are monitored during shear displacement, the use of a target pore-water pressure equalization value of 85% was considered fair and reasonable.

3.6 Interpretation of Data

The primary measurements obtained from the macro-scale interface direct shear test device are (i) the tensile force applied to the mobile frame; and (ii) the corresponding horizontal displacement of the mobile frame. In addition to these variables, the pore-water pressure at the test-soil/test-surface interface is measured when fully saturated (typically fine-grained soil) specimens are tested. These measurements are then used to obtain the variation of the interface shear stress with shear displacement during a given interface shear test, and in turn, to determine the interface shear strength of a given soil/solid interface. The following text provides the details of the data interpretation procedure.

A schematic free-body-diagram (FBD) of the testing system is illustrated in Fig. 3.25. The total weight of the system ($w_{total}$) comprises of the weights of the mobile frame ($w_{mf}$), the test soil layer ($w_{ts}$), the surcharge load sand layer ($w_{frs}$), the geocell grid ($w_{gc}$) and the non-woven geotextile ($w_{gt}$). The weight of the mobile frame ($w_{mf}$) is essentially transferred to the axles and four wheels that rest on support
3.6. Interpretation of Data

rails. This is based on the assumption that the friction between the soil materials contained within the mobile frame and the side-walls of the mobile frame is negligible, which is justified by the measures taken to reduce the side-wall friction of the mobile frame (using double-polyethylene lined walls as shown in Fig. 3.17). Therefore, the normal stress acting on the base plates ($\sigma_n$) is that produced by the test-soil and the applied surcharge stress. When the pulling force ($F_{pull}$) is applied to the mobile frame, a counteracting shear stress ($\tau_{total}$), which is comprised of the inherent friction of the moving parts of the mobile frame ($F_{df}$), hereafter referred to as the device friction, and the shear resistance at the interface, is produced. This shear force can be calculated as given by Eq. 3.3, using the pulling force measured by the load cell. Rearranging the terms in Eq. 3.3 results in Eq. 3.4, which is used to calculate the interface shear stress ($\tau_i$).

Figure 3.25: Schematic free body diagram of the mobile frame of the macro-scale interface direct shear test device.

\[
F_{pull} = \tau_{total}A_i = F_{df} + \tau_iA_i
\]  
(3.3)

\[
\tau_i = \frac{F_{pull} - F_{df}}{A_i}
\]  
(3.4)

In order to calculate the mobilized soil-interface shear stress ($\tau_i$), the device friction ($F_{df}$) is determined prior to conducting the interface shear test by translating the device without a soil specimen in place and recording the variation of the pulling force with horizontal displacement of the mobile frame. It was found that the inherent
3.6. Interpretation of Data

device friction was approximately 5% (or less) of the large-displacement pulling force measured during a typical macro-scale interface direct shear test.

The effective normal stress \( (\sigma'_n) \) acting on the interface at a given instant should be determined in the process of calculating test-soil/test-surface interface friction angle. This is accomplished by first calculating the total normal stress acting on the test surface as given by Eq. [3.1] and then deducting the average pore water pressure at the interface during shear as:

\[
\sigma'_n = \sigma_n - \frac{pp_1 + pp_2 + pp_3 + \ldots + pp_n}{n}
\]  

where, \( pp_1, pp_2, pp_3, \ldots, pp_n \) are pore-water pressure readings at the interface obtained from pressure sensors that are mounted on the test surface, and \( n \) is the total number of pressure sensors used. The interface friction angle \( (\phi'_i) \), is then calculated as per Eq. [3.6]. The pore-water pressure varies continuously as the soil specimen is sheared against the test surface. Hence these calculations are performed on each data point that is captured by the data acquisition system so that the variation of the interface shear strength with shear displacement can be obtained.

\[
\phi'_i = \arctan \left( \frac{\tau_i}{\sigma'_n} \right)
\]  

(3.6)
Chapter 4

Experimental Program and Results

This chapter presents the results from experiments performed to evaluate the overall performance of the newly built large-scale interface direct shear device. The capability of the device to determine the large displacement interface friction angle at a given test-soil/test-surface interface was evaluated by conducting several interface direct shear tests. A coarse-grained soil commonly used in geotechnical research at UBC was tested against a mild-steel surface at low effective normal stresses. The same interface was tested in a conventional direct shear apparatus under the same loading conditions in an attempt to compare the scale effects and performance of the devices. The results were compared with similar tests conducted using conventional interface shear devices. In addition, two fine-grained soils were tested under low effective normal stresses against the same mild-steel surface to evaluate the performance of the device in testing fine-grained soils. These tests were carried out with only three pore-water pressure transducers attached to the test plates. Subsequently, additional tests were conducted to determine if the device is capable of capturing the effect of a change of interface on the large-displacement interface shear strength at very low stresses by shearing three soils of different plasticity indices on an epoxy-coated mild steel surface under very low effective normal stresses. These tests were conducted with seven pore-water pressure transducers attached to the test plate. This provided the opportunity to investigate possible strain non-uniformities within the specimen during shear. The following sections provide details of these benchmark interface shear tests carried out using the macro-scale interface direct shear device.
4.1 Materials Tested

Three soil materials, namely, (i) Fraser River sand, (ii) a non-plastic silt obtained from a natural silt deposit, and (iii) a commercially available kaolinite clay were selected to use in the macro-scale interface direct shear tests. These three materials essentially represent the two ends of the grain size spectrum from coarse-grained to fine-grained soils. Details of the materials are described in the text that follows.

4.1.1 Fraser River sand

Fraser River sand, dredged from the Fraser River in the Lower Mainland of British Columbia, Canada, has been extensively used in element testing as well as soil-pipe interaction testing conducted at UBC over the past 10 years. Therefore, this material was selected as the benchmark coarse-grained test material. The Fraser River sand has an average particle size \( D_{50} = 0.26 \, \text{mm} \), \( D_{10} = 0.17 \, \text{mm} \), specific gravity \( G_s = 2.71 \), and uniformity coefficient \( C_u = 1.6 \). The grain size distribution of the sand is given in Fig. 4.1.

The maximum and minimum void ratios (\( e_{\text{max}} \) and \( e_{\text{min}} \)) for the sand determined
4.1. Materials Tested

as per American Society for Testing and Materials Standards ASTM-D4254 (2006) and ASTM-D4253-00 (2006) are 0.94 and 0.62, respectively. Fraser River sand is composed of 40% quartz, quartzite, and chert, 11% feldspar, and 45% unstable rock fragments, and 4% miscellaneous detritus (Garrison et al., 1969). The sand grains are generally angular to sub-rounded in shape. The material was tested against a bare mild-steel surface at effective normal stresses of approximately 3 kPa and 6 kPa. The sand was oven-dried and left to cool to room temperature prior to testing.

4.1.2 Non-plastic silt

A natural deposit of silt located at the south bank of the Fraser River adjacent to the Port Mann bridge in British Columbia was excavated to obtain the non-plastic silt material. The material was excavated using a commercial excavator and transported to UBC in 2000-lb bags. The silt is dark brown in color and has a fine-grit feel to the touch with the occasional coarse-gritty texture due to the presence of sand particles. The grain size distribution of the material is shown in Fig. 4.1. Index tests carried out on this material at an independent laboratory showed a natural moisture content of 23.2%, and a specific gravity, $G_s$, of 2.76. The material shows liquid and plastic limits of 28% and 27% respectively.

4.1.3 Kaolinite clay

A commercially available kaolinite material was purchased in powder form from the supplier - Unimin Corporation, USA. This clay material is yellowish-white in color and in its dry state resembles talcum powder. The chemical constituents of the material, as provided by the supplier, are listed in Table 4.1. The grain size distribution of the material is shown in Fig. 4.1. Index tests carried out on this clay material at an independent laboratory showed a specific gravity, $G_s$, of 2.66, and liquid and plastic limits of 48% and 26% respectively.
4.1. Materials Tested

Table 4.1: Chemical constituents of the kaolinite clay test-soil.

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Mean percentage by weight (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silicone dioxide (SiO₂)</td>
<td>45.9</td>
</tr>
<tr>
<td>Aluminium Oxide (Al₂O₃)</td>
<td>38.7</td>
</tr>
<tr>
<td>Titanium Dioxide (TiO₂)</td>
<td>1.70</td>
</tr>
<tr>
<td>Iron Oxide (Fe₂O₃)</td>
<td>0.30</td>
</tr>
<tr>
<td>Calcium Oxide (CaO)</td>
<td>0.03</td>
</tr>
<tr>
<td>Magnesium Oxide (MgO)</td>
<td>0.10</td>
</tr>
<tr>
<td>Potassium Oxide (K₂O)</td>
<td>0.10</td>
</tr>
<tr>
<td>Sodium Oxide (Na₂O)</td>
<td>0.08</td>
</tr>
<tr>
<td>Loss on ignition</td>
<td>13.7</td>
</tr>
</tbody>
</table>

4.1.4 Mild steel test surface

Two 7 mm thick hot-rolled mild-steel plates of type CS-Type-A (A1011/A1011M12b, 2012) each having dimensions of 1.52 m by 1.93 m in plan were used as the test-surface. The plates combined provided a plan area of approximately 6 m². A photograph of a small coupon of this type of mild-steel plate is shown in Fig. 4.2.

![Small coupon of mild-steel test surface](image)

Figure 4.2: Small coupon of mild-steel test surface used in the macro-scale interface direct shear tests.

The test-surface was smooth in texture by visual observation and the average
4.2 Tests Conducted

Surface roughness $R_a$ measured using a Mitutoyo-SJ210 type portable roughness tester in accordance with ISO4287-ISO/TC5 (1997) ranged from a minimum of 1.0 $\mu$m to maximum of 4.4 $\mu$m with an average of about 2.3 $\mu$m. The surface was mildly oxidized and the oxidation was found to rapidly spread across the surface within 24 hours after leaving the surface exposed to moisture. Hence, care was taken to keep the test-surface dry and clear of excessive oxidation whenever testing was not in progress. The test surface is cleaned and washed thoroughly with water and then dried well prior to each interface shear test.

4.1.5 Epoxy-coated mild steel test surface

Two 7 mm thick hot-rolled mild-steel plates of type CS-Type-A (A1011/A1011M12b, 2012) each having dimensions of 1.52 m by 1.93 m in plan were applied with 1 coat of a commercially available proprietary epoxy coating material supplied by a coating supplier in British Columbia, Canada. The steel base plates were coated professionally as per the suppliers specifications by a commercial organization specializing in epoxy coatings based in Surrey, BC, Canada. The coating was 0.15 cm thick and showed an average surface roughness Ra ranging from 0.10 m to 0.22 m with a mean of 0.16 m measured using a Mitutoyo-SJ210 type portable roughness tester in accordance with ISO4287-ISO/TC5 (1997).

4.2 Tests Conducted

A total of five different soil/solid interfaces were tested using the new device. A summary of the interfaces tested is given in Table 4.2. The tests involving the mild-steel surface were conducted to evaluate the ability of the device to test the large-displacement interface friction angle at very low effective normal stresses when a coarse-grained soil or a fine-grained soil is used. Usefulness of having pore-water pressure transducers attached to the test plate was investigated. The remainder of the tests involving the epoxy-coated mild steel surface were used to evaluate the performance of the device with respect to the effects of change in interface on the
4.3 Dry-Fraser-River-Sand / Mild-Steel Interface

Oven-dried Fraser River sand specimens were tested in the device against the surface of mild-steel. Drying of the sand had to be carried out in four batches over a period of four days due to the large volume of sand needed to prepare the specimen in the new device. Table 4.3 lists the details of each test in summary. Two of the four tests were conducted at approximately 6 kPa effective normal stress and the other two were conducted at approximately 3 kPa. The objective was to see if the device is capable of producing good quality results at large shear displacements under these low effective normal stresses. The tests also provided the opportunity to check the
4.3. Dry-Fraser-River-Sand / Mild-Steel Interface

repeatability of the tests. The next sub-section provides details of the experimental aspects and important details of the tests followed by the results and observations.

4.3.1 Details of tests

Each interface shear test was conducted by following the same test procedure from start to finish to minimize variability of the soil specimen and the surcharge load sand layer. The repeatability tests were conducted by preparing a new specimen in the mobile frame independent of the original test. The specimen preparation was carried out according to the procedure outlined in Section 3.4.1. The frictional resistance produced by the mechanical components of the device and the rubbing of the rubber wiper of the mobile frame against the test surface was measured prior to each test (see Fig. 4.3 for a typical device-friction response). The measured shear stress for the Fraser-River-sand/mild-steel interface is corrected for the device friction during processing of the data. Approximately 0.125 m$^3$ of oven-dried Fraser River sand that was cooled to room temperature was air-pluviated into the mobile frame in several batches from an elevation of approximately 1 m above the test surface and was levelled to form a 3 cm to 5 cm thick test specimen. Small containers of known volume were embedded in the specimen during filling so that the density of the sand could be determined before and after each shear test. An average unit weight of 14 kN/m$^3$ was observed for the test-soil specimen. The mild-steel test surface was cleaned well before placing the sand inside the shear device. The surcharge load of approximately 3 kPa was applied using the geo-cell reinforced sand layer as described in section 3.2.4. The surcharge load sand layer was moist with a moisture content of approximately 25% and showed an average moist unit weight of 16 kN/m$^3$.

The preparation of the test-soil specimen and surcharge loading took approximately 15 minutes and 2 hours respectively. Shearing was commenced immediately after the application of the surcharge load onto the test-soil specimen. The shearing of the specimen was carried out by applying a constant rate of shear displacement of 0.04 mm/s to the mobile frame. Each test was carried out until a shear displacement of about 400 mm was obtained. At the end of shear, the surcharge load
4.3. Dry-Fraser-River-Sand / Mild-Steel Interface

Table 4.3: Details of dry-Fraser-River-sand / mild-steel interface direct shear tests conducted on the macro-scale interface direct shear device.

<table>
<thead>
<tr>
<th>Test date</th>
<th>Average thickness of specimen (cm)</th>
<th>Average effective normal stress (kPa)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012 March 20</td>
<td>4.0</td>
<td>5.7</td>
<td>Evaluation of performance at the upper-bound of effective normal stress range.</td>
</tr>
<tr>
<td>2012 April 02</td>
<td>5.0</td>
<td>5.6</td>
<td>Repeatability test at the upper-bound of effective normal stress range.</td>
</tr>
<tr>
<td>2012 April 10</td>
<td>5.0</td>
<td>3.9</td>
<td>Evaluation of performance at the lower-bound of effective normal stress.</td>
</tr>
<tr>
<td>2012 April 12</td>
<td>4.0</td>
<td>3.0</td>
<td>Evaluation of performance at the lower-bound of effective normal stress.</td>
</tr>
</tbody>
</table>

Water was siphoned out and the geo-cell reinforced sand layer very carefully removed to expose the surface of the test specimen in an attempt to (i) visually observe the condition of the test soil specimen, (ii) to obtain the final thickness of the specimen, and (iii) to recover the embedded containers for density measurements. It was found to be very difficult to expose the test-soil surface without disturbing the specimen due to the nature of dry sand. Nevertheless careful excavation of the surcharge load sand layer allows for the estimation of the final thickness of the test-soil specimen with reasonable accuracy - the main intent being to check if the soil has maintained its volume intact over the 3 m² plan area.

4.3.2 Sources of error

Shear stress

The calculated shear stresses at large-displacement for the tests reported in the previous section are within the range of 1.5 kPa to 3.5 kPa. This corresponds to a shear force ranging between 4 kN and 10 kN measured by the load cell. The load cell re-
4.3. Dry-Fraser-River-Sand / Mild-Steel Interface

Figure 4.3: Variation of device friction with shear displacement measured by displacing the mobile frame with no soil specimen inside the mobile frame. (Device frictional resistance is the ratio of device friction force $F_{df}$ to interface shear area $A_i$).

Response within this range of loads is linear showing a calibration factor of 11.1 kN/V (per 10 V excitation). During testing, the data acquisition setup and wiring system used in these tests picks up a high frequency noise of approximately 3 Hz with average, maximum, and minimum amplitudes of 0.006 V, 0.04 V, and 0.001 V respectively. Due to this noise, the load cell readings are valid to a resolution of approximately 0.44 kN (i.e. 0.04 V x 11.1 kN/V). It was found that this noise is a result of the electro-magnetic field generated by the stepper motor that is picked up by the long lengths of wires that carry the data from the sensors to the data acquisition system. With the stepper motor turned off the system shows a resolution of 0.022 kN (i.e. 0.002 V x 11.1 kN/V). The noise that is due to the stepper motor has been filtered out in the results presented in the previous section. This translates to approximately $\pm 0.013$ kPa error in shear stress readings which is reflected in the results presented in the following sections.

**Effective normal stress**

Quality assurance measures were taken to ensure that the density of the surcharge load sand layer was determined at the end of each interface shear test. The density of the surcharge load sand layers showed a standard deviation of 0.125 kN/m$^3$. 

from the mean. This shows that the predicted effective normal stress can be off by approximately ±0.045 kPa.

### 4.3.3 Conventional direct shear testing

Interface shear strength results obtained from conventional direct shear apparatus under low effective normal stresses applicable to this study was not found in available literature. Therefore, a series of interface direct shear tests were conducted on the Fraser-River-sand/mild-steel interface utilizing one of the direct shear devices that are used in civil engineering undergraduate classes at UBC (see Fig. 4.4).

![Image of direct shear apparatus](image)

**Figure 4.4:** Photograph showing the components of the conventional direct shear apparatus used in undergraduate level laboratory classes at UBC.

This device carries a mobile frame that can be displaced horizontally at a constant rate using a stepper motor. The mobile frame holds the bottom half of the shear box.
in place and shearing of a soil specimen is done by displacing the mobile frame relative to the top shear box. The top shear box is held in a fixed position and hence, the horizontal displacement of the mobile frame causes shearing between the bottom and top shear boxes. A modification had to be made in order to utilize this device for interface shear testing such that it reproduces testing conditions that are similar to that of the macro-scale device. As shown in Fig. 4.5 the bottom shear box was replaced by a 7 cm thick mild-steel plate that measured 14 cm by 14 cm in plan.

Figure 4.5: Photograph showing the modified components of the conventional direct shear apparatus.

This plate was cut out of the same steel plate that was used in the macro-scale interface direct shear tests. The test plate was fixed firmly to the mobile frame so that the horizontal displacement of the mobile frame results in the horizontal displacement of the test plate. The top shear box was unaltered and attached to the fixed connection point located at the South end of the mobile frame as shown in Fig. 4.6. This fixed
connection point is directly attached to the horizontal load cell. The top shear box is
used to contain the test-soil specimen and creates an interface shear area of 100 cm²
(10 cm by 10 cm) in plan. A photograph of a soil specimen that was prepared inside
the top shear box is illustrated in Fig. 4.7.

Figure 4.6: Photograph showing the modified conventional direct shear apparatus
that was used for interface shear testing.

A metal top cap that measures slightly less than 10 cm by 10 cm in plan is placed
on top of the surface of the test soil specimen. The top cap has a ribbed surface where
it contacts the soil specimen and is used to (i) confine the soil specimen from the top,
(ii) provide a reference surface that can be used to measure the vertical displacement
of the surface of the test soil during shearing, and (iii) is used to support the surcharge
load that applies the desired effective normal stress onto the soil specimen (see Fig.
4.8).
4.3. Dry-Fraser-River-Sand / Mild-Steel Interface

Figure 4.7: Photograph showing the dry Fraser River sand specimen prepared inside the top shear frame of the modified conventional direct shear apparatus.

Figure 4.8: Photograph showing the top cap placed on top of the dry Fraser River sand specimen.
4.3. Dry-Fraser-River-Sand / Mild-Steel Interface

Figure 4.9: Photograph showing the surcharge load application mechanism on the modified conventional direct shear apparatus.

Standard weights placed on top of the top cap were used to apply the surcharge load. A standard 3 kg weight in combination of the weight of the top cap and the test-soil was sufficient to provide an effective normal stress of 3 kPa at the test-soil/test-surface interface (see Fig. 4.9). The soil specimen was sheared against the mild-steel surface as soon as the loading was applied. A shear displacement rate of 0.04 mm/s was used. Shearing was conducted until the maximum shear displacement attainable on the device was reached. The device was capable of a maximum shear displacement of 19 mm (approximately 127% of the thickness of the soil specimen).

4.3.4 Test results

The results of the macro-scale sand/mild-steel interface direct shear tests are presented in Fig. 4.10 and Table 4.4. The variation of the average interface shear stress \( \tau_i \) with shear displacement for all the tests seem to follow the same trend from start to end (see Fig. 4.10-a). For tests conducted at effective normal stresses within the 3 kPa to 4 kPa the interface shear stress rapidly reaches a peak followed by a slight de-
crease and then remains constant with further shear displacement. For the two tests conducted at effective normal stresses of approximately 6 kPa the shear stress rapidly reaches a plateau with no further change in shear stress with shear displacement.

![Graph showing shear stress vs shear displacement](image)

**Figure 4.10:** Fraser-River-sand/mild-steel macro-scale interface direct shear test results. (a) Variation of average shear stress with shear displacement. (b) Variation of normalized shear resistance with shear displacement.

The data presented in Fig. 4.10 shows that the average interface shear stress of the Fraser-River-sand/mild-steel interface, normalized to the effective normal stress, reaches a peak at a horizontal displacement of about 10 mm to 14 mm. This is approximately 2% to 3% of total shear displacement. Since the peak shear strength of a soil is sensitive to initial density, and can be affected by non-uniformities in strain...
4.3. Dry-Fraser-River-Sand / Mild-Steel Interface

development, it is often hard to confidently rely upon direct shear results for obtaining the peak shear strength. As such, it should be emphasized that the primary use of the macro-scale interface direct shear device is to obtain the large-displacement interface shear strength and not the peak shear strength. At displacements beyond about 25 mm (6% of total shear displacement), the shear resistance at the interface reaches a plateau and stays constant for the remaining duration of the test. This observation of unchanging shear stress at large displacements is in accord with the critical state condition expected when soils are subjected to large strains. Moreover, the essentially constant shear stress at large displacement observed in all tests, demonstrates that the new device is performing in accord with that typically expected from a direct shear test. A correlation of the mean interface shear stress at large displacement with average effective normal stress obtained from Fraser-River-sand/mild-steel macro-scale interface direct shear tests is presented in Fig. 4.11.

The results indicate that the normalized interface shear resistance, i.e. the average interface shear stress normalized by the effective normal stress is not significantly affected by the effective normal stress applied at the interface for the utilized effective normal stress range. In essence, a mean large displacement friction angle of 27.5 degrees (80% of the constant volume friction angle commonly reported for Fraser River sand) was observed for the Fraser-River-sand/mild-steel interface under effective
normal stresses within the range of 3 kPa to 6 kPa. The fact that the interface friction angle is less than the constant volume friction angle of Fraser River sand suggests that the shearing is not occurring within the sand specimen, but is either occurring at the interface between the soil and test surface or in a mechanism involving combined soil-soil internal shear and soil-solid interface shear.

Table 4.4: Test results of dry-Fraser-River-sand / mild-steel interface direct shear tests conducted on the macro-scale interface direct shear device.

<table>
<thead>
<tr>
<th>Effective normal stress at interface, $\sigma_n'$ (kPa)</th>
<th>Average interface shear stress at large-displacement, $\tau_i$ (kPa)</th>
<th>Macro-scale interface friction angle at large-displacement, $\phi_i = \tan^{-1}\left(\frac{\tau_i}{\sigma_n'}\right)$ (Degree)</th>
<th>Ratio of mean interface friction coefficient at large-displacement to constant-volume friction coefficient of test soil, $\frac{\tan(\phi_{i,\text{mean}})}{\tan(\phi_{cv})}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.67 ± 0.05</td>
<td>2.98 ± 0.01</td>
<td>27.7 ± 0.3</td>
<td>0.81</td>
</tr>
<tr>
<td>5.59 ± 0.05</td>
<td>2.91 ± 0.01</td>
<td>27.5 ± 0.3</td>
<td>0.80</td>
</tr>
<tr>
<td>3.86 ± 0.05</td>
<td>2.04 ± 0.01</td>
<td>27.9 ± 0.5</td>
<td>0.81</td>
</tr>
<tr>
<td>2.98 ± 0.05</td>
<td>1.56 ± 0.01</td>
<td>27.6 ± 0.5</td>
<td>0.81</td>
</tr>
</tbody>
</table>

4.3.5 Visual observations

It was observed that the test-soil together with the surcharge loading layers of reinforced sand and layer of water were well constrained within the mobile frame in all of the tests. At the South end of the mobile frame it was possible to observe a very thin trail of test soil that was left behind on the test-surface as the mobile frame progressed towards the North. The amount of test-soil that was left behind was insignificant (only about one grain size thick) and this observation was made in all of the tests that were conducted (see Fig. 4.12).

On the top surface of the surcharge load sand layer, a depression measuring approximately 10 cm wide and 2 cm deep was observed at the North end of the mobile frame (see Fig. 4.13). It was observed that this depression formed gradually during
the first 5 cm of shear displacement and remained constant thereafter. After approximately 5 cm of shear displacement this process subsides and no further change in surface elevation was observed with further shearing. Careful excavation of the surcharge load sand layer showed that the dry Fraser River sand specimen layer was intact across the whole 3 m² area. The minor depression at the North end of the mobile frame constitutes only 6% of the total interface shear area and its effect on effective normal stress when averaged over the total interface area is only 0.7% and hence, does not affect the calculated average interface shear strength.
4.3.6 Comparison with conventional direct shear testing

A total of four interface shear tests were conducted (two at 3 kPa effective normal stress and the others at 6 kPa) on the conventional direct shear apparatus. Fig. 4.14 shows the variation of the average interface shear stress $\tau_i$ with shear displacement for the dry-Fraser-River-sand/mild-steel interface obtained using the modified conventional direct shear apparatus. The results are summarised in Table 4.5. The variation of the average interface shear stress $\tau_i$ with shear displacement for all the tests showed the same trend from start to end consistent with results obtained from the macro-scale interface direct shear device at large displacements. For all tests the interface shear stress rapidly reached a peak followed by a gradual reduction until approximately 14 mm of shear displacement (93% of soil specimen thickness). After 14 mm shear
4.3. Dry-Fraser-River-Sand / Mild-Steel Interface

displacement the shear stress remained almost constant. The interface shear strength at 18 mm shear displacement was compared with the large-displacement macro-scale interface shear strength obtained from the macro-scale device.

Figure 4.14: Variation of average interface shear stress with shear displacement for dry-Fraser-River-sand/mild-steel interface obtained from the conventional direct shear device.

<table>
<thead>
<tr>
<th>Average effective normal stress at interface, $\sigma'_n$ (kPa)</th>
<th>Interface shear stress at 18 mm displacement, $\tau_i$ (kPa)</th>
<th>Interface friction angle at 18 mm displacement, $\phi_i$ (Degree)</th>
<th>Ratio of average interface friction coefficient to constant-volume friction coefficient of test soil ($\frac{\tan(\phi_i)}{\tan(\phi_{cv})}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.2</td>
<td>3.5 ± 0.1</td>
<td>29.4 ± 0.7</td>
<td>0.87</td>
</tr>
<tr>
<td>6.2</td>
<td>3.5 ± 0.1</td>
<td>29.4 ± 0.7</td>
<td>0.87</td>
</tr>
<tr>
<td>3.7</td>
<td>1.9 ± 0.1</td>
<td>27.2 ± 1.2</td>
<td>0.79</td>
</tr>
<tr>
<td>3.7</td>
<td>1.9 ± 0.1</td>
<td>27.2 ± 1.2</td>
<td>0.79</td>
</tr>
</tbody>
</table>

Comparison of the normalized interface shear resistance values obtained from the two devices shows that there is a significant amount of scatter in the data obtained from the conventional direct shear device (see Fig. 4.15). While the mean interface
friction angle obtained from the macro-scale device ranged from 27.5 degrees to 27.9 degrees with a 0.5 degree scatter about the mean, the same in the conventional direct shear data ranged from 27.2 degrees to 29.4 degrees with a 1.2 degree scatter about the mean. The quality of data obtained from the conventional apparatus is a function of the measurement equipment and the data acquisition system used. It should be noted that the scatter of the data may be reduced by using high precision instrumentation. However, it should be noted that the instrumentation used for measuring the shear force and the data acquisition system used in the conventional direct shear apparatus produced a resolution of 0.001 kN compared to 0.03 kN of the macro-scale device. Yet it is clear from Fig. 4.15 that the data obtained from the macro-scale device is more stable than that of the conventional device. For these tests the average interface shear stress measured on both devices is less than 4 kPa.

Figure 4.15: Variation of normalized interface shear stress with percentage shear displacement for dry-Fraser-River-sand/mild-steel interface obtained from the conventional direct shear device and the macro-scale interface direct shear device.

A shear stress of 4 kPa is reflected as a shear force of 0.04 kN on the conventional direct shear apparatus, and 12 kN on the macro-scale device. Given the resolution of 0.001 kN of the instrumentation used for measuring the shear force in the conven-
tional direct shear apparatus, measuring a force of 0.04 kN is completely within the acceptable capability of the device without loss of reliability. Yet a significant scatter in the data was observed. An attempt was not made to determine the source of the scatter. Careful analysis of the data shows that the largest peak-to-peak variation in the scatter of the data is approximately 0.4 kPa and corresponds to a shear force of $0.4 \text{kPa} \times 100 \text{m}^2 \times 10^{-4} = 0.004 \text{kN}$. This was found to be the same order of magnitude as the friction arising from the mechanical response of the small-scale device. Hence, it is possible that scatter in the data may be arising from the mechanical friction of the device. In comparison, the test data of the macro-scale device shows almost no scatter (see Fig. 4.15).

Table 4.6 compares results of these tests to those obtained from the macro-scale device. The interface shear strength at 18 mm shear displacement on the conventional device was compared with the large-displacement (400 mm displacement) interface shear strength obtained from the macro-scale device. Note that the friction between the top mobile frame of the conventional direct shear apparatus and the mild steel surface was determined prior to each test. This was accomplished by displacing the top mobile frame with no soil specimen in place across the mild steel surface and measuring the horizontal force with displacement and then performing the same test with the top mobile frame detached from the device so that the internal device friction can be measured.

The device friction in the conventional device was found to be between 21% and 37% of the measured shear force whereas for the macro-scale test it is between 1.7% and 3.2%. This clearly indicates that the macro-scale interface direct shear device is more suitable for testing soil-solid interfaces under such low effective stresses. While measures can be taken to reduce the device friction in the conventional direct shear apparatus the measured shear would still be within the same order of magnitude as the device friction and it is more reliable to use a large-scale device where the shear forces arising from material behaviour are much larger in comparison. The comparison shows that under very low effective normal stresses the macro-scale interface direct shear device is a better choice for producing high quality interface direct shear
Table 4.6: Results of dry-Fraser-River-sand / mild-steel interface direct shear tests conducted on the conventional direct shear device and the macro-scale interface direct shear device compared.

<table>
<thead>
<tr>
<th>Test</th>
<th>Average effective normal stress at interface, $\sigma'_n$ (kPa)</th>
<th>Measured shear force at 100 % displacement, $\tau_i$ (kN)</th>
<th>Ratio of device friction to measured shear force at 100 % displacement ($\frac{F_{df}}{F_{pull} - F_{df}} \times 100%$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CDS</td>
<td>6.2</td>
<td>0.034 ± 0.001</td>
<td>20.6</td>
</tr>
<tr>
<td>CDS</td>
<td>6.2</td>
<td>0.034 ± 0.001</td>
<td>20.6</td>
</tr>
<tr>
<td>CDS</td>
<td>3.7</td>
<td>0.019 ± 0.001</td>
<td>36.8</td>
</tr>
<tr>
<td>CDS</td>
<td>3.7</td>
<td>0.019 ± 0.001</td>
<td>36.8</td>
</tr>
<tr>
<td>MIST</td>
<td>5.67 ± 0.05</td>
<td>8.96 ± 0.03</td>
<td>1.67</td>
</tr>
<tr>
<td>MIST</td>
<td>5.59 ± 0.05</td>
<td>8.76 ± 0.03</td>
<td>1.71</td>
</tr>
<tr>
<td>MIST</td>
<td>3.86 ± 0.05</td>
<td>6.14 ± 0.03</td>
<td>2.44</td>
</tr>
<tr>
<td>MIST</td>
<td>2.98 ± 0.05</td>
<td>4.70 ± 0.03</td>
<td>3.19</td>
</tr>
</tbody>
</table>

CDS: Conventional  MIST: Macro-Scale

strength data.

The data obtained from the macro-scale device is suited for the intended purpose outlined in the beginning of the thesis due to several reasons:

1. The large 3 m² interface shear area provides the means to test interfaces under very low effective normal stresses. Effective normal stresses less than 10 kPa can be used without any loss of reliability of the data unlike in the case of the conventional direct shear device. The large shear area also minimizes, by averaging, local texture effects on the test surface that can affect results in a small-scale device.

2. Significantly large shear displacements can be achieved on the macro-scale device compared to the conventional device.

3. The device relies on a sand layer reinforced with a flexible cellular membrane to apply the normal stress to the specimen. Any stress non-uniformity that results
4.3. Dry-Fraser-River-Sand / Mild-Steel Interface

from the use of a rigid loading cap is eliminated.

4.3.7 Comparison with conventional interface shear test results reported by others

This section provides a brief comparison of interface shear strength results of various interfaces involving coarse-grained soils and steel surfaces of various surface roughness obtained from the use of conventional interface shear testing methods. In Table 4.7 the results of the macro-scale interface direct shear tests on dry-Fraser-River-sand/mild-steel interface are compared with those obtained from the conventional direct-shear apparatus by Karimian (2006) as well as from the ring-shear device (Rinne, 1989; Yoshimi and Kishida, 1981). Karimian (2006) reports findings of interface friction angles for dry-Fraser-River-sand/sand-blasted-mild-steel interfaces at effective normal stresses of 20 kPa and 37 kPa obtained using a conventional direct shear device that measures 75 mm by 75 mm in plan dimensions for the tested sand specimen. Rinne (1989) presents results of a series of ring-shear interface shear tests on dry-Target-sand/smooth-steel interfaces at effective normal stresses ranging between 100 kPa and 750 kPa. Rinne (1989)’s device accommodated an annular test soil specimen with inner and outer radii of 44.5 mm and 70 mm respectively. Yoshimi and Kishida (1981) present interface shear test results on dry-Tonegawa-sand/mild-steel interfaces carried out using a ring-shear device at an effective normal stress of 105 kPa. Their device produced annular test soil specimens 24 mm wide and 240 mm in inside diameter.

The comparison indicates that the interface friction angle for Fraser-River-sand/sand-blasted-steel (Karimian 2006) is greater than that for the Fraser-River-sand/smooth-steel interface obtained using the macro-scale device. It appears that the higher roughness of the sand-blasted steel surface in (Karimian, 2006) has contributed to the greater frictional resistance compared to that observed for smooth steel. The interface friction angle reported by Rinne (1989) for the Target-sand/smooth-steel interface is slightly lower than that obtained from the macro-scale interface shear
4.3. Dry-Fraser-River-Sand / Mild-Steel Interface

test. This could be attributable to the lower average surface roughness of the steel surface tested, which was 0.3 m, while that in the current study was 2.3 m. Yoshimi and Kishida (1981)’s work on Tonegawa sand has also shown dependence of the interface friction on the surface roughness of the steel surface. Unfortunately a direct comparison of their results with the macro-scale interface shear test cannot be done since the experimental conditions are not the same in the two cases.

Table 4.7: Test results of coarse-grained-soil / mild-steel interface shear tests conducted on conventional interface shear apparatus compared with the macro-scale interface direct shear results.

<table>
<thead>
<tr>
<th>Test description</th>
<th>Apparatus</th>
<th>Average effective normal stress $\sigma_n$ (kPa)</th>
<th>Average interface friction angle $\phi_i$ (Deg.)</th>
<th>Constant volume friction angle of soil $\phi_{cv}$ (Deg.)</th>
<th>$\frac{\tan(\phi_i)}{\tan(\phi_{cv})}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fraser-River-sand/sand-blasted-steel (Karimian 2006)</td>
<td>Conventional direct shear device</td>
<td>20</td>
<td>33</td>
<td>33</td>
<td>1.00</td>
</tr>
<tr>
<td>Fraser-River-sand/sand-blasted-steel (Karimian 2006)</td>
<td>Conventional direct shear device</td>
<td>37</td>
<td>30</td>
<td>33</td>
<td>0.91</td>
</tr>
<tr>
<td>Target-sand/smooth-steel (Rinne 1989)</td>
<td>UBC ring-shear device</td>
<td>100</td>
<td>23</td>
<td>33</td>
<td>0.70</td>
</tr>
<tr>
<td>Target-sand/smooth-steel (Rinne 1989)</td>
<td>UBC ring-shear device</td>
<td>500</td>
<td>20</td>
<td>33</td>
<td>0.61</td>
</tr>
<tr>
<td>Target-sand/smooth-steel (Rinne 1989)</td>
<td>UBC ring-shear device</td>
<td>750</td>
<td>20</td>
<td>33</td>
<td>0.61</td>
</tr>
<tr>
<td>Tonegawa-sand/smooth-steel, $R_a = 3.0 \mu m$</td>
<td>Ring-shear device</td>
<td>105</td>
<td>10</td>
<td>Not available</td>
<td>N/A</td>
</tr>
<tr>
<td>Tonegawa-sand/smooth-steel, $R_a = 5.0 \mu m$</td>
<td>Ring-shear device</td>
<td>105</td>
<td>20</td>
<td>Not available</td>
<td>N/A</td>
</tr>
<tr>
<td>Fraser-River-sand/smooth-steel, $R_a = 2.4 \mu m$ (Current study)</td>
<td>Macro-scale interface direct shear device</td>
<td>5.7</td>
<td>27.9</td>
<td>33</td>
<td>0.82</td>
</tr>
<tr>
<td>Fraser-River-sand/smooth-steel, $R_a = 2.4 \mu m$ (Current study)</td>
<td>Macro-scale interface direct shear device</td>
<td>3.0</td>
<td>28.1</td>
<td>33</td>
<td>0.82</td>
</tr>
</tbody>
</table>
4.3.8 Conclusion

The newly developed macro-scale interface direct shear test device in this thesis has produced high quality data for the large-displacement interface friction angle between dry Fraser River sand and mild steel under very low effective normal stresses. Moreover, the device shows very good repeatability of tests. The interface friction angle obtained immediately prior to reaching large displacements is not reliable due to (i) difficulty in accurately capturing the volume change behaviour of the specimen, and (ii) due to non-uniformities in the development of shear strain. The observed peaks may or may not be indicative of actual interface shear behaviour and may solely be attributed to phenomena that correspond to the rearrangement of the soil fabric to a stable state before shearing at the interface commences. The fact that the interface friction angle is less than the constant volume friction angle of Fraser River sand suggests that the shearing is not occurring within the sand specimen but is either occurring at the interface between the soil and test surface or in a complicated mechanism involving soil-soil internal shear and soil-solid interface shear. It is clear from these tests that the new device functions as intended with respect to coarse-grained soils sheared against a solid test surface.

4.4 Saturated-Non-Plastic-Silt / Mild-Steel Interface

Saturated non-plastic silt specimens were tested in the device against the surface of mild-steel at effective normal stresses of approximately 3 kPa and 6 kPa. Shearing was conducted at a shear displacement rate of 0.007 mm/s. A total of four independent tests were conducted at effective normal stresses within the 3 kPa to 6 kPa range. Table 4.8 lists the details of each test in summary. The objective was to see if the device is capable of producing good quality results at large shear displacements under these low effective normal stresses when a saturated fine-grained soil specimen is tested. The tests also provided the opportunity to check the repeatability of the
tests. The next sub-section provides details of the experimental aspects and important details of the tests followed by the results and observations.

### 4.4.1 Details of tests

Each interface shear test was conducted by following the same test procedure from start to finish. The specimen preparation was carried out according to the procedure outlined in the section 3.4.2. Approximately 0.3 m$^3$ of non-plastic silt slurry having a moisture content of 94% was allowed to flow into the mobile frame under gravity in several batches from an elevation of approximately 1 m above the test surface until a 10 cm thick test specimen was formed. Small containers of known volume were used to collect the slurry during filling so that the moisture content of the silt could be determined. After placement of the slurry inside the mobile frame it was left to consolidate under self-weight. Pore-water pressure at the base of the specimen was continually monitored. The mild-steel test surface was cleaned well before placing the silt specimen inside the shear device. The surcharge load of approximately 4.5 kPa was applied using the geo-cell reinforced sand layer as described in Section 3.2.4. The surcharge load sand layer was moist with a moisture content of approximately 24% and showed an average moist unit weight of 17 kN/m$^3$. In tests where an effective normal stress of 6 kPa to 7 kPa was required, the additional 2 kPa to 3 kPa was applied using a water mass in addition to the reinforced sand layer. It was difficult to obtain the moisture content of the self-weight consolidated specimen without disturbing the specimen and hence this moisture content is not known. However the moisture content at the end of shearing was obtained and this was used in the calculation of the final effective normal stress at the interface. An average unit weight of 19 kN/m$^3$ was observed for the test-soil specimen at end of shearing. Fig. 4.16 shows the time history of the pore-water pressure at the soil/solid interface that was recorded during application of surcharge load.

Note that only three pore-water pressure transducers, positioned in a straight line at the center of the mobile frame, were used in these tests. All three pore-water pressure transducer readings were found to read the same pore-water pressure after
Figure 4.16: Typical variation of average pore-water pressure at the test surface level observed during preparation of a silt specimen and surcharge loading.
the consolidation stage was complete. In several trial tests (not reported in this thesis for brevity), a few locations within the surcharge load layers were carefully excavated to expose the test soil specimen to observe the free water surface. Often the free water surface rose above the thickness of the test soil specimen and into the overlying surcharge load sand layer. Level measurements taken to the surface of the exposed free water surface revealed that the end-of-consolidation pore-water pressure was equal to the static head of water that is present above the test plate level. This supports the accuracy of the readings displayed by the pressure transducers.

### Table 4.8: Details of saturated-non-plastic-silt/mild-steel interface direct shear tests conducted on the macro-scale interface direct shear device.

<table>
<thead>
<tr>
<th>Test date</th>
<th>Average thickness of specimen at end of shearing (cm)</th>
<th>Average effective normal stress (kPa)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012 May 03</td>
<td>8.0</td>
<td>4.3</td>
<td>Evaluation of performance at the lower-bound of effective normal stress range.</td>
</tr>
<tr>
<td>2012 May 09</td>
<td>7.0</td>
<td>3.9</td>
<td>Evaluation of performance at the lower-bound of effective normal stress range.</td>
</tr>
<tr>
<td>2012 May 12</td>
<td>5.0</td>
<td>4.5</td>
<td>Repeatability test at the lower-bound of effective normal stress.</td>
</tr>
<tr>
<td>2012 May 16</td>
<td>4.0</td>
<td>5.9</td>
<td>Evaluation of performance at the upper-bound of effective normal stress.</td>
</tr>
</tbody>
</table>

#### 4.4.2 Test results

The results of the saturated-non-plastic-silt/mild-steel interface direct shear tests are presented in Figs. 4.17 | 4.21 and Table 4.9. The variation of pore-water pressure with shear displacement was measured during shearing and this allowed for the determination of effective normal stress continuously as the shearing was progressed. Fig. 4.17 shows plots of the variation of pore-water pressure with shearing and the
4.4 Saturated-Non-Plastic-Silt / Mild-Steel Interface

corresponding change in the average effective normal stress at the interface. The
effective normal stress thus calculated is used to determine the evolution of the aver-
age interface shear strength as shearing continues (see Fig. 4.21-b. The variation of
the average interface shear stress $\tau$ with shear displacement for all the tests seem to
follow the same trend from start to end (see Fig. 4.21-a). For all tests the interface
shear stress rapidly reaches a peak followed by a slight decrease and then gradually
increases until a plateau is reached at approximately 350 mm shear displacement
(500% of specimen thickness) and remains constant afterwards.

![Graph](image1)

**Figure 4.17:** Test results of macro-scale interface direct shear tests on saturated
non-plastic silt against a mild-steel test surface. (a) Variation of average pore-water
pressure at the interface with shear displacement. (b) Variation of average effective
normal stress at the interface with shear displacement.
4.4. Saturated-Non-Plastic-Silt / Mild-Steel Interface

The spacial configuration of the three pore-water pressure transducers relative to the displacing soil specimen changes continuously as shearing progresses. This allows for the observation of the pore-water pressure at the soil/solid interface along the center line of the specimen from North to South. Figs. 4.18 and 4.19 show the variation of pore-water pressure at individual pressure transducer locations observed in the tests. As shearing progresses the pressure transducer PP1 would measure the pore-water pressure very close to the South end of the mobile frame and eventually the soil specimen footprint would displace past the pressure transducer exposing it to the parts of the trailing end of the mobile frame and, eventually, to the atmosphere. For this reason, readings of PP1 were not used in the calculation of average pore-water pressure.

The pore water pressure response, as the specimen footprint was leaving PP1, also allowed to indirectly determine the region of influence of the end effects. Fig. 4.20 shows the region of influence of the end effects of the South and North sides of the mobile frame on the pore-water pressure at the soil/solid interface. Calculation of the pore-water pressure change within this region shows that it would amount to only a 2.4 % change in the pore-water pressure and hence was considered to be negligible. Therefore, the effective normal stress at the soil/solid interface was calculated based on the pore-water pressure readings of pressure transducers PP2, and PP3 assuming that it is the same over the whole footprint of the shear area. The zone of influence due to the boundary effects of the mobile frame is very small compared to the total area of the footprint of the soil/solid interface.

The results in Figs. 4.18, 4.20 and 4.19 show that at large shear displacements the pore-water pressure at the soil/solid interface reaches a stable state. Theoretically, a change in pore-water pressure at the soil/solid interface can be observed if the fine-grained soil experiences any volumetric strain tendencies. Hence, a stable pore-water pressure state that is measured by the pore-water pressure transducers at the soil/solid interface at large displacements would represent a stable state of the interface shearing process. The primary purpose of commissioning of a new device that was capable of producing large shear displacements was to provide the means to determine the
Figure 4.18: Variation of pore-water pressure with shear displacement observed during macro-scale interface direct shear tests on saturated non-plastic silt against a mild-steel test surface conducted on May/12/2012.
Figure 4.19: Variation of pore-water pressure with shear displacement observed during macro-scale interface direct shear tests on saturated non-plastic silt against a mild-steel test surface conducted on May/16/2012.
4.4. Saturated-Non-Plastic-Silt / Mild-Steel Interface

Figure 4.20: Variation of pore-water pressure with shear displacement observed during macro-scale interface direct shear tests on saturated non-plastic silt against a mild-steel test surface conducted on May/09/2012.
4.4. Saturated-Non-Plastic-Silt / Mild-Steel Interface

interface shear strength at such large displacement stable states that are possible to achieve only at relatively large shear displacements.

The variation of the average shear stress and the calculated interface shear resistance are shown in Fig. 4.21. Table 4.9 provides a summary of the results. A correlation of the mean interface shear stress at large displacement with average effective normal stress is presented in Fig. 4.22. An average interface friction angle of 25.6 degrees was observed for this interface under the low effective normal stresses tested.

Figure 4.21: Test results of macro-scale interface direct shear tests on saturated non-plastic silt against a mild-steel test surface. (a) Variation of average shear stress with shear displacement. (b) Variation of normalized shear resistance with shear displacement.
### 4.4. Saturated-Non-Plastic-Silt / Mild-Steel Interface

Table 4.9: Test results of macro-scale interface direct shear tests conducted on saturated-non-plastic-silt/mild-steel interface.

<table>
<thead>
<tr>
<th>Average effective normal stress at large displacement ($\sigma'_n$, kPa)</th>
<th>Average Interface Shear Stress at 500 mm Displacement ($\tau'_i$, kPa)</th>
<th>Interface friction angle at 500 mm shear displacement ($\phi_i$, Degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.28 ± 0.05</td>
<td>2.13 ± 0.01</td>
<td>26.4 ± 0.4</td>
</tr>
<tr>
<td>3.92 ± 0.05</td>
<td>1.95 ± 0.01</td>
<td>26.4 ± 0.4</td>
</tr>
<tr>
<td>4.57 ± 0.05</td>
<td>2.27 ± 0.01</td>
<td>26.4 ± 0.4</td>
</tr>
<tr>
<td>5.85 ± 0.05</td>
<td>2.88 ± 0.01</td>
<td>26.2 ± 0.3</td>
</tr>
</tbody>
</table>

Figure 4.22: Mean interface shear stress at large displacement versus average effective normal stress obtained from saturated-non-plastic-silt/mild-steel macro-scale interface direct shear tests.

Fig. 4.23 shows the variation of the normalized interface shear resistance obtained from macro-scale interface direct shear tests on saturated-non-plastic-silt/mild-steel and dry-Fraser-River-sand/mild-steel interfaces for comparison. This comparison indicates that the normalized interface shear resistance for the non-plastic-silt/mild-steel interface is about 7% lower than that for the dry-Fraser-River-sand/mild-steel interface.
4.4. Saturated-Non-Plastic-Silt / Mild-Steel Interface

Figure 4.23: Variation of normalized interface shear resistance with shear displacement obtained from macro-scale interface direct shear tests on saturated-non-plastic-silt/mild-steel and dry-Fraser-River-sand/mild-steel interfaces compared.

The tests conducted at approximately the same effective normal stress (i.e. $\sigma'_n$ of 4.3 kPa, and 4.5 kPa) showed very good repeatability. Pore-water pressure measurements during the consolidation stage can be effectively used to determine a suitable shear displacement rate that would ensure minimal excess pore-water pressure generation during testing. The results also showed that the measurement of pore-water pressure during testing can be quite useful in obtaining a more comprehensive understanding of the evolution of the interface shear resistance.

4.4.3 Visual observations

It was observed that the test-soil together with the surcharge loading layers of reinforced sand and layer of water were well constrained within the mobile frame in all of the tests. At the South end of the mobile frame it was possible to observe the traces of test soil that was left behind on the test-surface as the mobile frame progressed towards the North. The amount of test-soil that was left behind was insignificant and this observation was made in all of the tests that were conducted (see Fig. 4.24).

The surcharge load water layer and surcharge load sand layer were carefully removed at the end of each test to expose the test soil surface. This allowed for the
determination of the thickness of the silt specimen at the end of shearing and allowed for collection of soil samples for moisture content calculations. A photograph of the exposed silt specimen surface at the end of shearing is shown in Fig. 4.25. The silt specimen was found to be coherent and without cracks and of constant thickness throughout the whole footprint. Minor depressions of 5 cm width were visible at the North, South, East, and West edges of the specimen (see Fig. 4.26).

It is unclear what causes these depressions at the four edges of the specimen. A possible explanation can be arrived at by considering the drainage conditions at the edges of the specimen. It is likely that at the edges of the specimen there are two drainage boundaries as opposed to the only one drainage boundary in interior area of the specimen. It is possible that the drainage of pore-water is more rapid at the edges compared to the interior of the specimen and hence larger consolidation settlement is observed at the edges. But with no pore-water pressure transducers attached very close to the edges there is no conclusive evidence that supports this view. Further experimentation could be carried out to gain a better understanding of this process. Also a hypothetical flow-net diagram can be used to calculate the effective normal stress non uniformity close to the edges of the specimen to obtain a more accurate value for the effective normal stress. However considering the overall scale of the specimen these edge effects can be regarded as insignificant and are not pursued in this thesis.
Figure 4.25: Photograph showing the top of the mobile frame at the end of a non-plastic-silt/mild-steel interface direct shear test after carefully removing the surcharge load layers to expose the silt specimen.
Figure 4.26: Photograph showing the depression of the silt specimen at the North edge observed at the end of a non-plastic-silt/mild-steel interface direct shear test after carefully removing the surcharge load layers to expose the silt specimen.
4.5 Saturated-Non-Plastic-Silt / Epoxy-Coated-Mild-Steel Interface

A saturated non-plastic silt specimen was tested in the new device against the surface of epoxy-coated-mild-steel at an total normal stress of approximately 7 kPa. Shearing was conducted at a shear displacement rate of 0.007 mm/s. Shearing was stopped at a total shear displacement of approximately 800 mm. The objective was to investigate if the device is capable of effectively distinguishing between the interface shear strengths mobilized at large shear displacement for different types of soil/solid interfaces at very low effective normal stresses. Also a total of six pore-water pressure transducers were used to capture the spacial variability of the pore-water pressure over a large portion of the footprint of the interface.

4.5.1 Details of test

The specimen preparation was carried out according to the procedure outlined in Section 3.4.2. Approximately 0.45 m$^3$ of non-plastic silt slurry having a moisture content of 150% was allowed to flow into the mobile frame under gravity in several batches from an elevation of approximately 1 m above the test surface until a 15 cm thick test specimen was formed. Small containers of known volume were used to collect the slurry during filling so that the moisture content of the silt could be determined. After placement of the slurry inside the mobile frame it was left to consolidate under self-weight. The pore-water pressure at the base of the specimen was continually monitored through the use of six pore-water pressure transducers attached to the test plate (see Fig. 4.27). Note that there were seven pressure transducers attached to the device, but one of the transducers (PP7) malfunctioned during this testing work.

Fig. 4.27 shows the time history of the pore-water pressure at the base of the soil specimen during application of surcharge load and subsequent consolidation stage. Application of surcharge load takes approximately 4 hours to complete. During this
4.5. Saturated-Non-Plastic-Silt / Epoxy-Coated-Mild-Steel Interface

Figure 4.27: Variation of pore-water pressure at the test surface observed during preparation of the silt specimen on the epoxy-coated mild steel surface.

time the pore-water pressure at the soil/solid interface increases as the surcharge load increases. Note that the application of surcharge load is done manually by hand. Hence, a uniform distribution of surcharge load over the entire area of the specimen cannot be expected. This is reflected in the pore-water pressure readings. However, upon completion of the application of surcharge load, the excess pore-water pressure that was generated during the surcharge loading stage, starts to dissipate in a uniform manner. This indicates that the pore-water pressure distribution at the soil/solid interface is quite uniform during the consolidation stage. Note that pressure transducers $PP1$, $PP2$, and $PP3$, are located approximately at the center of the
footprint of the soil specimen and the remaining pressure transducers are located very close to the North end of the mobile frame. Assuming symmetrical boundary conditions, it is clear from the pressure transducer readings that the pore-water pressure distribution at the soil/solid interface over is uniform during the consolidation stage. Consolidation of the specimen was allowed to progress for approximately 18 hours. The change in level of the surcharge load sand layer was measured to determine the change in thickness of the silt specimen during the consolidation process. Shearing was commenced afterwards.

### 4.5.2 Test results

The variation of pore-water pressure transducer readings at the soil/solid interface are shown in Fig. 4.28. A rapid rise in pore-water pressure at the soil/solid interface was observed at the start of shearing until a peak is reached at a shear displacement of 8 mm. Afterwards, the pore-water pressure begins to decrease with continued shearing. The rapid rise in pore-water pressure observed in the initial stage of shearing can be attributed to the volume change tendencies experienced by the soil as shearing commences.

The spacial variation of the pore-water pressure transducer readings within the first 100 mm of shear displacement does not appear to be uniform over the footprint of the specimen. The rise in pore-water pressure recorded by the transducers that are close to the North end of the mobile frame is much higher than that recorded by the transducers near the center of the footprint of the mobile frame. It is highly likely that this is due to the effects of progressive shear behaviour of the soil specimen at the soil/solid interface. However it is quite difficult to explain the behaviour with any certainty due to the complexity in the phenomena that may be taking place during the initial stage of shearing. After approximately 100 mm shear displacement, the spacial distribution of the pore-water pressure at the soil/solid interface becomes uniform. All pressure transducers begin show approximately the same pore-water pressure. This suggests that after approximately 100 mm shear displacement, the volume change tendencies of the soil specimen at the soil/solid interface subside and a stable state of...
Figure 4.28: Variation of pore-water pressure at the soil/solid interface with shear displacement observed during the non-plastic-silt/epoxy-coated-mild-steel interface direct shear test conducted on the macro-scale device.

interface shear is reached. Note that the pore-water pressure continues to gradually decrease with further shear displacement. It is highly likely that this is due to some drainage that was observed to be occurring at the four corners of the mobile frame. The rubber seals that seal drainage at the North, South, East, and West sides of the mobile frame do not effectively seal drainage at the four corners of the mobile frame. These observations of variation of the pore-water pressure at the soil/solid interface show that the pore-water pressure measurement system used in the current study has very low compliance. Any change in the static head of pore-fluid at the soil/solid interface is immediately reflected in the transducer readings. The steady decrease in the pore-water pressure (due to water draining out) at the soil/solid interface results in a steady rise in the effective normal stress. This is reflected as a steady rise in the...
measure shear force.

The continuous measurement of pore-water pressure at the soil/solid interface enables the determination of the variation of the effective normal stress at the soil/solid interface as shearing progresses. The variation of the average interface shear stress with shear displacement and the variation of the normalized interface shear resistance of the non-plastic-silt/epoxy-coated-mild-steel interface are shown in Fig. 4.29.

![Graph of average interface shear stress vs. shear displacement](image1)

![Graph of normalized interface shear resistance vs. shear displacement](image2)

Figure 4.29: Test results of the non-plastic-silt/epoxy-coated-mild-steel interface direct shear test conducted on the macro-scale device. (a) Variation of the average interface shear stress with shear displacement. (b) Variation of the normalized interface shear resistance with shear displacement.

An interface friction angle of 22.8 degrees (normalized interface shear resistance of 0.42) was obtained for the non-plastic-silt/epoxy-coated-mild-steel interface at a displacement of 800 mm. The calculated effective normal stress at the soil/solid
interface at 800 mm shear displacement was 3.67 kPa ± 0.05 kPa.

4.6 Saturated-Kaolinite-Clay / Mild-Steel Interface

A saturated kaolinite clay specimen was tested in the device against the surface of mild-steel at an effective normal stress of approximately 4.3 kPa. Shearing was conducted at a shear displacement rate of 0.00044 mm/s. Shearing was stopped at a total shear displacement of 280 mm. The objective was to see if the device is capable of producing good quality results at large shear displacements under low effective normal stresses when a saturated clay specimen is tested.

4.6.1 Details of test

The specimen preparation was carried out according to the procedure outlined in Section 3.4.2. Approximately 0.45 m³ of kaolinite clay slurry having a moisture content of 150% was allowed to flow into the mobile frame under gravity in several batches from an elevation of approximately 1 m above the test surface until a 15 cm thick test specimen was formed. Small containers of known volume were used to collect the slurry during filling so that the moisture content of the silt could be determined. After placement of the slurry inside the mobile frame it was left to consolidate under self-weight. Pore-water pressure at the base of the specimen was continually monitored (see Fig. 4.30).

The mild-steel test surface was cleaned well before placing the clay specimen inside the shear device. The surcharge load of approximately 3.2 kPa was applied using the geo-cell reinforced sand layer as described in Section 3.2.4. The surcharge load sand layer was slightly moist with a moisture content of approximately 24% and showed an average moist unit weight of 17 kN/m³. It was difficult to obtain the moisture content of the self-weight consolidated clay specimen without disturbing the specimen and hence this moisture content is not known. However the moisture content at the
4.6. Saturated-Kaolinite-Clay / Mild-Steel Interface

Figure 4.30: Variation of average pore-water pressure at the test surface level observed during preparation of kaolinite clay specimen and surcharge loading.

end of shearing was obtained and this was used in the calculation of the final effective normal stress at the interface. An average unit weight of 16 kN/m$^3$ was observed for the test-soil specimen at end of shearing.

4.6.2 Test results

The results of the saturated-kaolinite-clay/mild-steel interface direct shear test are presented in Figs. 4.31, and 4.32.

The variation of pore-water pressure with shear displacement was measured during shearing and this allows for the determination of effective normal stress continuously as the shearing is progressed. Fig. 4.31 shows plots of the variation of pore-water pressure with shearing and the corresponding change in the average effective normal stress at the interface. The effective normal stress thus calculated is used to determine the evolution of the average interface shear strength as shearing continues (see Fig. 4.32-b). The variation of the average interface shear stress $\tau_i$ with shear displacement is shown in Fig. 4.32-a. The interface shear stress gradually increases until a plateau is reached at approximately 150 mm shear displacement (100% of specimen thickness) and remains more or less constant afterwards. Based on the pore-water pressure variation observed during the test it is clear that the excess pore-water pressure had
4.6. Saturated-Kaolinite-Clay / Mild-Steel Interface

Figure 4.31: Test results of macro-scale interface direct shear test on saturated kaolinite clay against a mild-steel test surface. (a) Variation of average pore-water pressure at the interface with shear displacement. (b) Variation of average effective normal stress at the interface with shear displacement.

not completely dissipated at the end of shearing. It is preferable that the interface shear strength is obtained after the excess pore-water pressure has been completely dissipated.

Based on past tests on fine-grained soils this requires at least 400 mm to 500 mm of shear displacement. However this test was stopped at 280 mm displacement due to time constraints - a shear displacement of around 800 mm requires approximately one month to complete at the displacement rate used. Nevertheless the pore-water pressure variation data provides the means to calculate the effective normal stress as
4.6. Saturated-Kaolinite-Clay / Mild-Steel Interface

Figure 4.32: Test results of macro-scale interface direct shear test on saturated kaolinite clay against a mild-steel test surface. (a) Variation of average shear stress with shear displacement. (b) Variation of normalized shear resistance with shear displacement.

The test progresses and hence provided the necessary information for calculation of the interface shear strength. A normalized interface shear resistance of 0.51 was observed at 280 mm shear displacement (approximately 187% of specimen thickness). This corresponds to an interface friction angle of 27.0 degrees. The calculated effective normal stress at the soil/solid interface at 280 mm shear displacement was 4.57 kPa ± 0.05 kPa.
4.6.3 Visual observations

It was observed that the test-soil together with the surcharge loading layers of reinforced sand and layer of water were well constrained within the mobile frame in all of the tests. At the South end of the mobile frame it was possible to observe the trail of test soil that was left behind on the test-surface as the mobile frame progressed towards the North. The amount of test-soil that was left behind was insignificant and this observation was made in all of the tests that were conducted in this research program.

Figure 4.33: Photograph showing the top of the mobile frame at the end of the kaolinite-clay/mild-steel interface direct shear test after carefully removing the surcharge load layers to expose the clay specimen.

The surcharge load water layer and surcharge load sand layer were carefully removed at the end of each test to expose the test soil surface. This allowed for the
determination of the thickness of the clay specimen at the end of shearing and allowed for collection of soil samples for moisture content calculations. A photograph of the exposed clay specimen surface at the end of shearing is shown in Fig. 4.33. The clay specimen was found to be very well intact and of constant thickness throughout the whole 3-m² area. Minor depressions of 5 cm width were visible at the North, South, East, and West edges of the specimen similar to that observed in the non-plastic-silt/mild-steel interface shear tests.

4.7 Saturated-Kaolinite-Clay / Epoxy-Coated-Mild-Steel Interface

In order to further assess the effectiveness of the macro-scale interface direct shear test device, an interface direct shear test was performed using reconstituted saturated kaolinite clay on the epoxy-coated mild steel surface. The main purpose of this test was to determine the large-displacement interface friction angle between kaolinite and epoxy-coated-mild-steel at effective normal stresses of approximately 3 kPa. The results of the test are intended for demonstrating (i) the performance of the new device in testing saturated fine-grained soils against a solid surface for the interface shear resistance at large displacements; and (ii) to demonstrate the use of the new device in capturing the dependence of the interface shear strength on the type of interface tested.

4.7.1 Details of test

Specimen preparation was carried out as outlined in Section 3.4.2. The variation of pore-water pressure at the soil/solid interface during application of surcharge load and subsequent consolidation are shown in Fig. 4.34. Note that pore-water pressure transducer PP3 was not functioning during this test. Application of surcharge load takes approximately 3 to 4 hours to complete. During this time the pore-water pressure at the soil/solid interface increases as the surcharge load increases. Note
that the application of surcharge load is done manually by hand. Hence, a uniform distribution of surcharge load over the entire area of the specimen cannot be expected. Yet, great care was taken to apply the surcharge load uniformly over the test soil specimen. This is evident from the observed uniform increase in the pore-water pressure shown in Fig. 4.34.

The sudden “break” in the pore water pressure rise at around 2.5 kPa in Fig. 4.34 was due to the placement of sand being temporarily stopped during the swapping of containers that are used to store the sand material - two containers, each holding approximately 0.4 m$^3$ of sand are usually required to prepare the surcharge sand.
layer. Once the sand containers were swapped, the application of surcharge load was continued until the desired amount of surcharge load was reached. Upon completion of the application of surcharge load, the excess pore-water pressure that was generated during the surcharge loading stage, starts to dissipate in a uniform manner. This indicates that the pore-water pressure distribution at the soil/solid interface is quite uniform during the consolidation stage. The observed variation of the pore-water pressure transducer readings at the soil solid interface are consistent with that observed during the specimen preparation stage of the non-plastic-silt/epoxy-coated-mild-steel interface shear test described in Section 4.5. Consolidation of the specimen was allowed to progress for approximately four days. The change in level of the surcharge load sand layer was measured to determine the change in thickness of the clay specimen during the consolidation process. Shearing was commenced afterwards.

4.7.2 Test results

The results of the saturated-kaolinite-clay/epoxy-coated-mild-steel interface direct shear test are presented in Figs. 4.35 and 4.36. As shown in Fig. 4.36-(a), the average shear stress reaches a mild peak at approximately 50 mm shear displacement (about 500% of specimen thickness) followed by a gradual drop and then remains stable for the remainder of the test. As shown in Fig. 4.35 the pore-water pressure at the soil/solid interface increase rapidly as shearing commences, and reaches a peak at around 10 mm shear displacement (about 100% of specimen thickness), then continuously decreases with increasing shear displacement until it stabilizes at approximately 400 mm shear displacement (about 4000% of specimen thickness). Fig. 4.35 clearly shows that at large displacements there is almost no spatial variability of the pore-water pressure within the footprint of the shear area. It is evident from this that, at large displacements, the boundary effects on the pore-water pressure distribution within the specimen is minimal and suggests that the larger footprint of the device effectively minimizes, by averaging, the potential end effects at the North, and South ends of the specimen.

The consistent variation of all the pore-water pressure transducer readings ob-
4.7. Saturated-Kaolinite-Clay / Epoxy-Coated-Mild-Steel Interface

Figure 4.35: Variation of pore-water pressure at the soil/solid interface with shear displacement observed during the kaolinite-clay/epoxy-coated-mild-steel interface direct shear test conducted on the macro-scale device.

served in all tests that involved saturated fine-grained soils, further demonstrates good repeatability of measurements. This information allows for the determination of the evolution of the effective normal stress at the interface quite effectively and this is very important in the accurate determination of the interface shear strength. It is also of interest to note that, as shown in Fig. 4.36(b), the normalized interface shear resistance reaches a plateau at around 400 mm shear displacement (at approximately 4000% of specimen thickness). A normalized interface shear resistance of 0.25 was observed thereafter. This corresponds to an interface friction angle of 14.0 degrees at 500 mm shear displacement. The calculated effective normal stress at the soil/solid interface at 500 mm shear displacement was 4.34 kPa ± 0.05 kPa.
Figure 4.36: Test results of the kaolinite-clay/epoxy-coated-mild-steel interface direct shear test conducted on the macro-scale device. (a) Variation of the average interface shear stress with shear displacement. (b) Variation of the normalized interface shear resistance with shear displacement.

## 4.8 Conclusion

The shear strength results of soil/solid interfaces that were tested on the macro-scale interface direct shear device are summarized in Table [4.10]. Fraser-River sand on the mild steel surface showed the highest average interface shear strength of 27.7 degrees at large shear displacement, and kaolinite on the epoxy-coated mild steel surface showed the lowest value of 14.0 degrees. The published data available for the interface friction between clay/epoxy-coated-steel interfaces, from an effective stress point of
view, is scarce. This is primarily due to the reason that the characterization of shear strength of clayey materials are commonly accounted using total stress, instead of effective stress approaches (i.e., undrained shear strength instead of effective friction angle). It is of interest to investigate the effects of effective normal stress on the interface shear resistance.

Table 4.10: Comparison of macro-scale soil/solid interface direct shear test results of interfaces tested.

<table>
<thead>
<tr>
<th>Interface Tested</th>
<th>Shear Displacement at which Interface Shear Strength is Evaluated, $x$ (mm)</th>
<th>Effective normal stress at interface at $x$ shear displacement, $\sigma'_n$ (kPa)</th>
<th>Macro-scale interface friction angle at $x$ shear displacement, $\phi_i = \tan^{-1}\left(\frac{\tau_i}{\sigma'_n}\right)$ (Degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fraser-River-sand / mild-steel</td>
<td>400</td>
<td>2.98 ± 0.05</td>
<td>27.6 ± 0.5</td>
</tr>
<tr>
<td>Fraser-River-sand / mild-steel</td>
<td>400</td>
<td>3.86 ± 0.05</td>
<td>27.9 ± 0.5</td>
</tr>
<tr>
<td>Fraser-River-sand / mild-steel</td>
<td>400</td>
<td>5.59 ± 0.05</td>
<td>27.5 ± 0.3</td>
</tr>
<tr>
<td>Fraser-River-sand / mild-steel</td>
<td>400</td>
<td>5.67 ± 0.05</td>
<td>27.7 ± 0.3</td>
</tr>
<tr>
<td>Non-plastic-silt / mild-steel</td>
<td>500</td>
<td>3.92 ± 0.05</td>
<td>26.4 ± 0.4</td>
</tr>
<tr>
<td>Non-plastic-silt / mild-steel</td>
<td>500</td>
<td>4.28 ± 0.05</td>
<td>26.4 ± 0.4</td>
</tr>
<tr>
<td>Non-plastic-silt / mild-steel</td>
<td>500</td>
<td>4.57 ± 0.05</td>
<td>26.4 ± 0.4</td>
</tr>
<tr>
<td>Non-plastic-silt / mild-steel</td>
<td>500</td>
<td>5.85 ± 0.05</td>
<td>26.2 ± 0.3</td>
</tr>
<tr>
<td>Kaolinite / mild-steel</td>
<td>280</td>
<td>4.57 ± 0.05</td>
<td>27.0 ± 0.3</td>
</tr>
<tr>
<td>Non-plastic-silt / epoxy-coated-mild-steel</td>
<td>800</td>
<td>3.67 ± 0.05</td>
<td>22.8 ± 0.3</td>
</tr>
<tr>
<td>Kaolinite / epoxy-coated-mild-steel</td>
<td>500</td>
<td>4.34 ± 0.05</td>
<td>14.0 ± 0.4</td>
</tr>
</tbody>
</table>

Based on the data summarized by Mitchell and Soga (2005), the effective friction angle of reconstituted, normally consolidated pure kaolinite would range between 25.5
4.8. Conclusion

to 39.0. The interface friction angle of 14.0 observed for the kaolinite clay/epoxy-coated-steel interface from the current testing is significantly below the lower end of the range of internal friction angles reported for kaolinite by Mitchell and Soga (2005). This clearly suggests that the shearing is occurring at the soil/epoxy-coated-steel interface, in turn, resulting in a significantly lower effective friction angle than the internal friction angle of kaolinite. These tests clearly demonstrate the performance of the new device with respect to testing of saturated fine-grained soils for the large-displacement drained interface friction angle.

It is interesting to note that the kaolinite, tested on the mild steel surface, showed an interface shear strength almost the same as that observed for the Fraser-River sand on the mild steel interface. It was also observed that the average interface shear strength of the non-plastic silt on the mild steel surface approximately the same as that observed for the Fraser-River sand on the mild steel interface. Yet the same non-plastic silt material showed a lower large-displacement interface shear strength when tested on the epoxy-coated mild steel surface (see Fig. 4.37). This clearly shows that the device is capable of capturing the dependence of the large-displacement interface shear strength on the type of interface tested under very low effective normal stress conditions.

Figure 4.37: Variation of normalized interface shear resistance with shear displacement observed for some of the soil/solid interfaces tested on the macro-scale device.
4.8. Conclusion

The newly developed macro-scale interface direct shear test device in this thesis has produced high quality data for the large-displacement interface friction angle between various soils ranging from coarse-grained to fine-grained and mild steel under very low effective normal stresses. Moreover, the device shows consistent repeatability of measurements as evidenced from the test results covered in Sections 4.4–4.5. The total of nine tests that were conducted to evaluate the performance of the new device showed that the device has the capability to:

- test fine-grained or coarse-grained soils against solid surfaces commonly used in the industry;
- perform interface direct shear tests at effective normal stresses ranging from 3 kPa to 6 kPa;
- achieve a maximum shear displacement of 1.2 m;
- impart very low shear displacement rates (0.0001 mm/s to 1 mm/s) to ensure pore-water pressure stabilization at the soil/solid interface when saturated fine-grained soils are tested;
- monitor pore-water pressure variation during shearing and hence the ability to determine the effective normal stress at the soil/solid interface.

The device has the following several unique features:

1. The large 3 m$^2$ interface shear area provides the means to test interfaces under very low effective normal stresses. Effective normal stresses less than 10 kPa can be used without any loss of reliability of the data, unlike in the case of a conventional direct shear device. The large shear area also minimizes, by averaging, local texture effects on the test surface that can affect results in a small-scale device.

2. The device is equipped with seven pore-water pressure transducers. These enable the determination of pore-water pressure at the interface and hence the
4.8. Conclusion

effective normal stress in real-time during testing. The large shear area aids in minimizing end-effects and leads to a very good estimation of the average effective normal stress. Based on the observation of the variation of pore-water pressure near the South boundary of the mobile frame, it was shown in Section 4.4 that, the pore-water pressure change due to the boundary effects of the mobile frame within the influence region would amount to only a 2.4 % and hence was considered to be negligible. The pore-water pressure effects close to the boundary of the device can be significant in small-scale devices whereas in a large device this can be averaged over the large area without compromising the interpretation of results. It is uncommon that consideration is given to such details in conventional direct shear testing.

3. Pore-water pressure measurements during the consolidation stage can be effectively used to determine a suitable shear displacement rate that would ensure minimal excess pore-water pressure generation during testing. The results also showed that the measurement of pore-water pressure during testing can be quite useful in obtaining a clear knowledge of the variation of the effective normal stress at the interface.

4. The device relies on a sand layer reinforced with a flexible cellular membrane to apply the normal stress to the specimen. Any stress non-uniformity that results from the use of a rigid loading cap is eliminated.

As for any test device, there are several limitations and practical difficulties that need to be appreciated in considering this macro-scale direct shear test device for obtaining interface friction angles at soil/solid interfaces:

1. The apparatus is not considered suitable for estimating the peak friction angle considering the high potential for strain non-uniformities across the specimen in the early stages of shear displacement.

2. It is relevant to note that the pore water pressure measurement is an important requirement when testing the interface friction against fine grained soils.
4.8. Conclusion

Moreover, when testing clay/solid interfaces, the tests have to be performed at extremely low displacement time rates. For the kaolinite tested in this research, it was found that a displacement rate of 0.0004 mm/s is suitable for stabilization of pore-water pressure at the soil/solid interface at a displacement of 300 mm. Although a detailed study was not performed to evaluate the rate dependency of the interface shear strength, occasional trial tests were performed on the kaolinite/mild-steel interface at displacement rates within the range of 0.01 mm/s to 1 mm/s. An increase in the interface shear strength was observed when displacement rates within the above range was used. However, it should be noted that these observations were made from tests that were not performed in a controlled manner and hence should be treated with caution.

3. The use of a layer of soil for the application of surcharge load on the test soil specimen can introduce an error in the predicted surcharge load. This is mainly due to the inherent variability of the density of the surcharge load soil layer that results from manual filling. Special care and quality control measures need to be employed to ensure that the density variation is eliminated. Use of uniformly-sized lead spheres to form the surcharge load layer may be a suitable alternative.

4. One of the other disadvantages is that the specimen preparation is time consuming and it involves significant physical effort.

It should be noted that this device is not the only device currently available that can produce high-quality data for use in aiding the development of fundamentals of the problem of soil-pipeline interaction in the offshore environment. The conventional direct shear and ring-shear devices as well as the tilt-table test are viable options. However, there are inherent limitations in these conventional devices that make their use in testing soil/solid interfaces under very low effective normal stresses quite difficult without special modifications. The new device has been shown to work well under these special testing conditions and contributes to the field of offshore pipeline
4.8. Conclusion

gotechnics by supplementing a new test method to the array of conventional testing apparatus that are already available.
Bibliography


123


Bibliography


126


