PIEZOCONE TECHNOLOGY FOR THE GEOENVIRONMENTAL CHARACTERIZATION OF MINE TAILINGS

by

Michael Paul Davies, P.Eng., P.Geo.

B.A.Sc. (Hons.) Brit. Col. 1985

A THESIS SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY
in
THE FACULTY OF GRADUATE STUDIES
Department of Civil Engineering

We find the thesis meets the required standards
THE UNIVERSITY OF BRITISH COLUMBIA
©Michael Paul Davies, 1999
In presenting this thesis in partial fulfilment of the requirements for an advanced degree at the University of British Columbia, I agree that the Library shall make it freely available for reference and study. I further agree that permission for extensive copying of this thesis for scholarly purposes may be granted by the head of my department or by his or her representatives. It is understood that copying or publication of this thesis for financial gain shall not be allowed without my written permission.

Department of Civil Engineering

The University of British Columbia
Vancouver, Canada

Date 30 August 99
ABSTRACT

The mining industry produces large volumes of milled mineral tailings. These tailings are separated/removed from ore by mechanical, chemical and/or biological means during processing sequences aimed at isolating the economic commodity. In many cases, particularly in metaliferous and oil sands mining, the economic commodity is a very small amount of the total mass of ore processed. Modern mining is increasingly processing greater tonnages as the economies of small-scale mining decrease. It is often the case that the tonnage of ore processed is essentially equal to the tonnage of tailings produced and, consequently, mine tailings storage facilities are often required to be very large.

Mine tailings storage facilities often represent the single largest environmental liability with the mining process. To design and operate against the large number of potential physical and chemical failure modes, designers and operators of tailings storage facilities must adequately characterize the nature of the tailings within these facilities and then manage the given facility accordingly. The large size of many of these facilities makes adequate characterization a challenging task. As if to add credibility to the nature of this challenge, of the roughly two major tailings storage facility failures per year over the past four decades, many of the failures have been directly attributed to a lack of understanding of the in-situ character of the given tailings.

Piezocone testing, carried out in-situ under prevailing physico-chemical stresses, was assessed to see if it could offer any advantageous technology for the geoenvironmental characterization of mine tailings. In particular, an assessment of new developments in resistivity piezocone technology were applied to mine tailings for the first time to see if there was any ability to characterize these materials for a very broad range of key physical and chemical traits.

Besides a general assessment of the suitability of piezocone technology for characterizing mine tailings, the research included more in-depth assessments of several key aspects of the geoenvironmental character of mine tailings. In the process of these assessments, several modest contributions appear to have been developed. Amongst these original contributions are:

- introduction of the resistivity piezocone to mine tailings including new generation resistivity modules with isolated measurement technology for linearity of calibration and greater range and reliability of response;
- introduction of the concept of a material index for assessing the soil behavior type of mine tailings;
- introduction of a fines content assessment procedure from material index;
- development of a method to estimate tailings compressibility from material index;
- suggested approach for estimating static and cyclic liquefaction susceptibility directly from piezocone tests using concepts of material state;
- rationalization of constrained versus unconstrained undrained strength of liquefied tailings and the introduction of piezocone-direct techniques for estimating the appropriate undrained strength in either case;
- introduction of a case history of a mine tailings static liquefaction event;
- a suggested approach to estimate hydraulic conductivity anisotropy of tailings deposits;
- introduction of resistivity piezocone technology to the assessment of sulphide tailings deposits including assessment of several ionic strength-bulk conductivity trends;
- a suggested normalization technique for bulk resistivity measurements; and
an initial effort at combining/comparing electromagnetic signatures from surface commercial techniques with subsurface resistivity piezocone measurements.

From an overall geotechnical contribution, the research also adds to the relatively sparse literature on cohesionless silt-sized soils. Mine tailings typically have unique fabric, gradation, grain angularity and stress-history when compared to most cohesionless soil deposits. Understanding these differences is important when “importing” traditional soil mechanics to mine tailings.

The research work supporting the thesis included over 200 piezocone soundings from 8 minesites and 4 well-characterized tool calibration sites in a wide variety of physico-chemical conditions. Over 100 pore water samples with geochemical analyses complimented this piezocone database. In addition to introducing existing and proposed evaluative procedures, the research database was used for verification of the proposed procedures. Part of this verification process included clearly identifying differences between mine tailings and natural mineral soils and these differences can affect traditional piezocone interpretive techniques; e.g. the use of shear wave velocity to predict in-situ state.

The main conclusion developed from this research is that current piezocone technology, when applied within an appropriate assessment framework, can characterize to a screening level many aspects of the geoevironmental nature of a wide variety of mine tailings. Several new procedures for estimating key geoevironmental parameters of mine tailings are provided. Additional research is required to determine which of these procedures may be globally tenable and to what degree the procedures require modification.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>II</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>X</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>XI</td>
</tr>
<tr>
<td>LIST OF SYMBOLS</td>
<td>XVI</td>
</tr>
<tr>
<td>ACKNOWLEDGMENTS</td>
<td>XVIII</td>
</tr>
<tr>
<td>1. INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 General</td>
<td>1</td>
</tr>
<tr>
<td>1.2 This Research</td>
<td>3</td>
</tr>
<tr>
<td>1.3 This Thesis</td>
<td>4</td>
</tr>
<tr>
<td>1.4 Engineering Disclaimer</td>
<td>7</td>
</tr>
<tr>
<td>2. GEOENVIRONMENTAL SITE CHARACTERIZATION</td>
<td>9</td>
</tr>
<tr>
<td>2.1 General</td>
<td>9</td>
</tr>
<tr>
<td>2.2 Basic Definitions</td>
<td>9</td>
</tr>
<tr>
<td>2.3 Characterization Philosophy</td>
<td>10</td>
</tr>
<tr>
<td>2.4 Requirements for Mine Tailings Projects</td>
<td>16</td>
</tr>
<tr>
<td>3. MINE TAILINGS</td>
<td>17</td>
</tr>
<tr>
<td>3.1 Overview</td>
<td>17</td>
</tr>
<tr>
<td>3.2 General</td>
<td>17</td>
</tr>
<tr>
<td>3.3 Historical Perspective</td>
<td>18</td>
</tr>
<tr>
<td>3.4 Tailings Characteristics</td>
<td>22</td>
</tr>
<tr>
<td>3.4.1 Types of Tailings</td>
<td>22</td>
</tr>
<tr>
<td>3.4.2 Depositional Characteristics</td>
<td>24</td>
</tr>
<tr>
<td>3.4.3 Typical Physical Properties</td>
<td>26</td>
</tr>
<tr>
<td>3.4.4 Typical Geochemistry</td>
<td>27</td>
</tr>
<tr>
<td>3.5 Tailings Storage Options</td>
<td>28</td>
</tr>
<tr>
<td>3.5.1 General</td>
<td>28</td>
</tr>
<tr>
<td>3.5.2 Types of Tailings Dams</td>
<td>28</td>
</tr>
</tbody>
</table>
# TABLE OF CONTENTS

(continued)

<table>
<thead>
<tr>
<th>PAGE</th>
<th>4. PIEZOCONE TECHNOLOGY</th>
</tr>
</thead>
<tbody>
<tr>
<td>34</td>
<td>4.1 Overview</td>
</tr>
<tr>
<td>34</td>
<td>4.2 In-Situ Testing</td>
</tr>
<tr>
<td>34</td>
<td>4.3 Traditional Tailings Characterization Methods</td>
</tr>
<tr>
<td>38</td>
<td>4.4 Standard Piezocone Technology</td>
</tr>
<tr>
<td>38</td>
<td>4.4.1 General</td>
</tr>
<tr>
<td>42</td>
<td>4.4.2 Interpretation of Data</td>
</tr>
<tr>
<td>47</td>
<td>4.4.3 Typical Engineering Uses</td>
</tr>
<tr>
<td>49</td>
<td>4.4.4 Piezocones Used in This Study</td>
</tr>
<tr>
<td>51</td>
<td>4.5 Seismic Piezocone</td>
</tr>
<tr>
<td>54</td>
<td>4.6 Resistivity Piezocone</td>
</tr>
<tr>
<td>54</td>
<td>4.6.1 General Concept</td>
</tr>
<tr>
<td>57</td>
<td>4.6.2 Development of the Research Tool Used</td>
</tr>
<tr>
<td>63</td>
<td>4.6.3 Evolution of the RCPTU During this Research</td>
</tr>
<tr>
<td>66</td>
<td>4.6.4 Calibration</td>
</tr>
<tr>
<td>78</td>
<td>4.6.5 Typical Values</td>
</tr>
<tr>
<td>82</td>
<td>4.6.6 Additional Electromagnetic Information</td>
</tr>
<tr>
<td>84</td>
<td>4.7 Fast Penetration and Cyclic Piezocone</td>
</tr>
<tr>
<td>89</td>
<td>4.8 Piezocone Scale Issues</td>
</tr>
<tr>
<td>93</td>
<td>4.9 Pore Fluid Sampling</td>
</tr>
<tr>
<td>93</td>
<td>4.9.1 Commercial BAT Sampling</td>
</tr>
<tr>
<td>95</td>
<td>4.9.2 KBAT</td>
</tr>
<tr>
<td>96</td>
<td>4.10 Surface Electromagnetic Methods</td>
</tr>
<tr>
<td>102</td>
<td>4.11 Summary of In-Situ Technologies Used in This Research</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>PAGE</th>
<th>5. MINE TAILINGS DATABASE</th>
</tr>
</thead>
<tbody>
<tr>
<td>103</td>
<td>5.1 General</td>
</tr>
<tr>
<td>104</td>
<td>5.2 Main Projects</td>
</tr>
<tr>
<td>104</td>
<td>5.2.1 General</td>
</tr>
</tbody>
</table>
# TABLE OF CONTENTS

(continued)

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.2.2</td>
<td>MENDO-INCO</td>
<td>106</td>
</tr>
<tr>
<td>5.2.3</td>
<td>SCBC</td>
<td>107</td>
</tr>
<tr>
<td>5.2.4</td>
<td>CANLEX</td>
<td>110</td>
</tr>
<tr>
<td>5.3</td>
<td>Other Contributions to Database</td>
<td>112</td>
</tr>
<tr>
<td>6.</td>
<td>SOIL BEHAVIOR TYPE</td>
<td>113</td>
</tr>
<tr>
<td>6.1</td>
<td>Overview</td>
<td>113</td>
</tr>
<tr>
<td>6.2</td>
<td>Perspective</td>
<td>113</td>
</tr>
<tr>
<td>6.3</td>
<td>Existing Methods for Estimating Soil Behaviour Type</td>
<td>114</td>
</tr>
<tr>
<td>6.4</td>
<td>Concept of Material Index</td>
<td>122</td>
</tr>
<tr>
<td>6.5</td>
<td>Evaluation of Material Index</td>
<td>126</td>
</tr>
<tr>
<td>6.6</td>
<td>Fines Content</td>
<td>132</td>
</tr>
<tr>
<td>7.</td>
<td>IN-SITU STATE</td>
<td>140</td>
</tr>
<tr>
<td>7.1</td>
<td>Overview</td>
<td>140</td>
</tr>
<tr>
<td>7.2</td>
<td>Background</td>
<td>140</td>
</tr>
<tr>
<td>7.3</td>
<td>Estimating In-Situ State of Mine Tailings</td>
<td>147</td>
</tr>
<tr>
<td>7.4</td>
<td>Basic Validation of Methodology</td>
<td>151</td>
</tr>
<tr>
<td>7.5</td>
<td>In-Situ Assessment of Methodology</td>
<td>157</td>
</tr>
<tr>
<td>7.6</td>
<td>Tailings Database Validation of the Methodology</td>
<td>163</td>
</tr>
<tr>
<td>7.7</td>
<td>Additional Comments</td>
<td>163</td>
</tr>
<tr>
<td>8.</td>
<td>STATIC LOAD GEOTECHNICAL PARAMETERS</td>
<td>170</td>
</tr>
<tr>
<td>8.1</td>
<td>Overview</td>
<td>170</td>
</tr>
<tr>
<td>8.2</td>
<td>Piezocone for Estimating Parameters</td>
<td>170</td>
</tr>
<tr>
<td>8.3</td>
<td>Correlation between the Piezocone and Standard Penetration Test</td>
<td>171</td>
</tr>
<tr>
<td>8.3.1</td>
<td>Proposed Methodology</td>
<td>171</td>
</tr>
<tr>
<td>8.3.2</td>
<td>Tailings Database Evaluation of Proposed Methodology</td>
<td>176</td>
</tr>
<tr>
<td>8.4</td>
<td>Static Drained Parameters</td>
<td>182</td>
</tr>
<tr>
<td>8.5</td>
<td>Static Undrained Parameters</td>
<td>183</td>
</tr>
<tr>
<td>8.6</td>
<td>Summary</td>
<td>184</td>
</tr>
</tbody>
</table>
# TABLE OF CONTENTS
(continued)

<table>
<thead>
<tr>
<th>SECTION</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.</td>
<td>TRANSIENT LOAD GEOTECHNICAL PARAMETERS</td>
<td>185</td>
</tr>
<tr>
<td>9.1</td>
<td>Overview</td>
<td>185</td>
</tr>
<tr>
<td>9.2</td>
<td>Definitions</td>
<td>186</td>
</tr>
<tr>
<td>9.3</td>
<td>Small Strain Dynamic Modulus and Damping</td>
<td>187</td>
</tr>
<tr>
<td>9.4</td>
<td>The Liquefaction Phenomena</td>
<td>194</td>
</tr>
<tr>
<td>9.4.1</td>
<td>General</td>
<td>194</td>
</tr>
<tr>
<td>9.4.2</td>
<td>Liquefaction and Mine Tailings</td>
<td>195</td>
</tr>
<tr>
<td>9.5</td>
<td>Assessing Liquefaction Susceptibility</td>
<td>202</td>
</tr>
<tr>
<td>9.5.1</td>
<td>General</td>
<td>202</td>
</tr>
<tr>
<td>9.5.2</td>
<td>Traditional SPT Approach</td>
<td>204</td>
</tr>
<tr>
<td>9.5.3</td>
<td>Piezocone Tip Stress Approaches</td>
<td>208</td>
</tr>
<tr>
<td>9.5.4</td>
<td>Shear Wave Velocity Approaches</td>
<td>209</td>
</tr>
<tr>
<td>9.5.5</td>
<td>Non-Piezocone In-Situ Methods</td>
<td>215</td>
</tr>
<tr>
<td>9.5.6</td>
<td>Integrated Piezocone Approach</td>
<td>217</td>
</tr>
<tr>
<td>9.6</td>
<td>Sample Liquefaction Susceptibility Screening - Integrated Piezocone Approach</td>
<td>218</td>
</tr>
<tr>
<td>9.6.1</td>
<td>Overview</td>
<td>218</td>
</tr>
<tr>
<td>9.6.2</td>
<td>Beaufort Sea Hydraulic Fills</td>
<td>220</td>
</tr>
<tr>
<td>9.6.3</td>
<td>Sullivan Tailings</td>
<td>221</td>
</tr>
<tr>
<td>9.6.4</td>
<td>Alaskan Tailings</td>
<td>223</td>
</tr>
<tr>
<td>9.6.5</td>
<td>Mildred Lake Settling Basin</td>
<td>228</td>
</tr>
<tr>
<td>9.6.6</td>
<td>J-Pit CANLEX Trial</td>
<td>231</td>
</tr>
<tr>
<td>9.6.7</td>
<td>Syncrude SWSS Facility</td>
<td>231</td>
</tr>
<tr>
<td>9.7</td>
<td>Operative Undrained Strength</td>
<td>235</td>
</tr>
<tr>
<td>9.7.1</td>
<td>Overview</td>
<td>235</td>
</tr>
<tr>
<td>9.7.2</td>
<td>Stress Level Dependent - Constrained Analyses</td>
<td>238</td>
</tr>
<tr>
<td>9.7.3</td>
<td>Practical Implications of Constrained Undrained Strengths</td>
<td>255</td>
</tr>
<tr>
<td>9.7.4</td>
<td>Flowslide Development - Unconstrained Analyses</td>
<td>259</td>
</tr>
<tr>
<td>9.7.5</td>
<td>Sullivan Tailings Flowslide - Case Example of Static Liquefaction</td>
<td>265</td>
</tr>
</tbody>
</table>
# TABLE OF CONTENTS (continued)

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.7.6 Moshikoshi Tailings Flowslide - Case Example of Seismic Liquefaction</td>
<td>276</td>
</tr>
<tr>
<td>9.7.7 Cyclic Piezocone</td>
<td>279</td>
</tr>
<tr>
<td>9.7.8 Summary</td>
<td>286</td>
</tr>
<tr>
<td>10. HYDROGEOLOGICAL PARAMETERS</td>
<td>289</td>
</tr>
<tr>
<td>10.1 Modeling Requirements</td>
<td>289</td>
</tr>
<tr>
<td>10.2 Water Table and Gradients</td>
<td>290</td>
</tr>
<tr>
<td>10.3 Hydraulic Conductivity</td>
<td>296</td>
</tr>
<tr>
<td>10.3.1 General</td>
<td>296</td>
</tr>
<tr>
<td>10.3.2 Saturated Values</td>
<td>301</td>
</tr>
<tr>
<td>10.3.3 Pore Pressure Dissipation Methods</td>
<td>302</td>
</tr>
<tr>
<td>10.3.4 Direct Relationships to Mechanical Piezocone Data</td>
<td>306</td>
</tr>
<tr>
<td>10.3.5 Direct Relationships to Bulk Resistivity Measurements</td>
<td>309</td>
</tr>
<tr>
<td>10.3.6 KBAT</td>
<td>309</td>
</tr>
<tr>
<td>10.3.7 Anisotropy</td>
<td>313</td>
</tr>
<tr>
<td>10.4 Degree of Saturation</td>
<td>318</td>
</tr>
<tr>
<td>11. GEOCHEMICAL NATURE OF PORE FLUIDS</td>
<td>322</td>
</tr>
<tr>
<td>11.1 General</td>
<td>322</td>
</tr>
<tr>
<td>11.2 Resistivity as an Indicator of Geochemistry</td>
<td>323</td>
</tr>
<tr>
<td>11.3 Data Repeatability</td>
<td>324</td>
</tr>
<tr>
<td>11.4 Sulphide Mine Tailings</td>
<td>328</td>
</tr>
<tr>
<td>11.4.1 General</td>
<td>328</td>
</tr>
<tr>
<td>11.4.2 ARD Processes</td>
<td>330</td>
</tr>
<tr>
<td>11.4.3 Resistivity Piezocone in Sulphide Tailings Environments</td>
<td>334</td>
</tr>
<tr>
<td>11.5 Global Relationships</td>
<td>347</td>
</tr>
<tr>
<td>11.6 Proposed Normalizing Procedure</td>
<td>361</td>
</tr>
<tr>
<td>12. RCPTU IN RELATIONSHIP TO SURFACE ELECTRO-MAGNETICS</td>
<td>364</td>
</tr>
<tr>
<td>12.1 General</td>
<td>364</td>
</tr>
<tr>
<td>12.2 EM31 Data</td>
<td>365</td>
</tr>
</tbody>
</table>
TABLE OF CONTENTS
(continued)

<table>
<thead>
<tr>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.3 Creating Conductivity Files from RCPTU Files</td>
</tr>
<tr>
<td>12.4 Discussion</td>
</tr>
<tr>
<td>13. SUMMARY AND RECOMMENDED FURTHER STUDY</td>
</tr>
<tr>
<td>13.1 Summary</td>
</tr>
<tr>
<td>13.1.1 General</td>
</tr>
<tr>
<td>13.1.2 Piezocone Technology</td>
</tr>
<tr>
<td>13.1.3 Tailings Database</td>
</tr>
<tr>
<td>13.1.4 Soil Behaviour Type</td>
</tr>
<tr>
<td>13.1.5 Geotechnical Parameters</td>
</tr>
<tr>
<td>13.1.6 Hydrogeological Parameters</td>
</tr>
<tr>
<td>13.1.7 Pore Fluid Geochemistry</td>
</tr>
<tr>
<td>13.2 Recommendations for Further Study</td>
</tr>
<tr>
<td>REFERENCES</td>
</tr>
</tbody>
</table>

LIST OF APPENDICES

Appendix I Summary Site Descriptions and Sample Data
Appendix II Surface Electromagnetics in Relation to RCPTU - Computer Program
LIST OF TABLES

Table 3.1 Typical Physical Parameters of Various Mine Tailings ........................................... 27
Table 3.2 Comparison of Tailings Dam Types ........................................................................ 32
Table 5.1 Key References to Mine Tailings Research Piezocone Database .................................. 103
Table 5.2 Funding Arrangement for SCBC Project .................................................................. 108
Table 5.3 SCBC Field Program ............................................................................................... 109
Table 5.4 SCBC Project Statistics - Piezocone Technology ..................................................... 109
Table 5.5 SCBC Project Statistics - Supporting Technologies ................................................ 110
Table 6.1 Soil Behaviour Type from Material Index $I_c$ ............................................................ 123
Table 6.2 Summary of Material Indices for a Range of Tailings Materials .............................. 131
Table 7.1 Calculation of $Q_p$ and F for Representative Values of $(N_1)_{60-ECS}$ ................... 155
Table 7.2 Approximate Equivalence Between $\Psi$ and SPT $(N_1)_{60-ECS}$ for Tailings with $I_c < 2.5$ ........................................................... 157
Table 8.1 Estimated Static Geotechnical Parameters for Tailings Using Established Relationships for General Soils ................................................................. 183
Table 9.1 Typical Values of Low Strain Damping Ratio for Standard Soils and from Tailings Database and Related ISTG Information ......................................................... 193
Table 9.2 Specific Gravities for Tailings Materials ................................................................. 219
Table 9.3 Undrained Residual Strength Data for Several Case Histories ............................... 240
Table 9.4 Recommended Approach for Estimating Undrained Strength of Liquefied Tailings for Stability Assessments Using SPT Data (modified from Davies and Campanella, 1994) ............................................................ 253
Table 9.5 Proposed Screening Level Estimation Relationships for Stress Normalized Undrained Residual Strength ................................................................. 254
Table 9.6 Summary of CANLEX Laboratory Testing on Syncrude Tailings - Steady State Results ............................................................................................................ 254
Table 10.1 Average Value of Bulk Hydraulic Conductivity for Typical Mine Tailings .......... 301
Table 10.2 Example of Piezocone Dissipation Data from Tailings Database - Sullivan Old Iron Pond ............................................................................................................. 305
Table 10.3 Results of Proposed Method for Estimated $K$ Anisotropy ..................................... 318
Table 11.1 Changes in Water Chemistry During Acid Generation ........................................ 332
Table 11.2 Examples of Acid Rock Drainage Quality (adapted from Fytas et al., 1992) ......... 334
### LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Recommended Minimum Sampling Quantity versus Size of Unit (adapted from BC Acid Drainage Task Force, 1989)</td>
</tr>
<tr>
<td>3.1</td>
<td>Typical Tailings Gradations (adapted from Klohn, 1995)</td>
</tr>
<tr>
<td>3.2</td>
<td>Methods of Tailings Dam Construction</td>
</tr>
<tr>
<td>3.3</td>
<td>Comparison of Fill Volumes for Tailings Dam Construction Methods (adapted from Vick, 1990)</td>
</tr>
<tr>
<td>4.1</td>
<td>Cut-Away Schematic of Modern Piezocone</td>
</tr>
<tr>
<td>4.2</td>
<td>Typical Arrangement for Conducting a SCPTU</td>
</tr>
<tr>
<td>4.3</td>
<td>Concept of Resistivity/Conductivity (adapted from McNeill, 1980)</td>
</tr>
<tr>
<td>4.4</td>
<td>Schematic of RES001</td>
</tr>
<tr>
<td>4.5</td>
<td>Schematic of RES002</td>
</tr>
<tr>
<td>4.6</td>
<td>Schematic of RES003</td>
</tr>
<tr>
<td>4.7</td>
<td>Resistivity Module Calibration Tank</td>
</tr>
<tr>
<td>4.8</td>
<td>Effect of Temperature on Pore Fluid Conductivity</td>
</tr>
<tr>
<td>4.9</td>
<td>Typical Calibration Data from RES001</td>
</tr>
<tr>
<td>4.10</td>
<td>Typical Calibration Data from RES002</td>
</tr>
<tr>
<td>4.11</td>
<td>Typical Low Excitation Calibration Data from RES003</td>
</tr>
<tr>
<td>4.12</td>
<td>Typical Medium Excitation Calibration Data from RES003</td>
</tr>
<tr>
<td>4.13</td>
<td>Typical High Excitation Calibration from RES003</td>
</tr>
<tr>
<td>4.14</td>
<td>Expanded Scale High Excitation Calibration from RES003</td>
</tr>
<tr>
<td>4.15</td>
<td>Schematic Bulk Resistivity-Conductivity Response Freshwater Sand Deposit</td>
</tr>
<tr>
<td>4.16</td>
<td>Concept of Induced Polarization (adapted from Beck, 1981)</td>
</tr>
<tr>
<td>4.17</td>
<td>Idealized Stress Conditions for Soil Element Adjacent to Piezocone</td>
</tr>
<tr>
<td>4.18</td>
<td>Schematic of Modified Friction Sleeve for Cyclic Piezocone Testing</td>
</tr>
<tr>
<td>4.19</td>
<td>Conceptual Depth of Influence for Bulk Resistivity Measurement</td>
</tr>
<tr>
<td>4.20</td>
<td>Commercial BAT Pore Fluid Sampling Tool</td>
</tr>
<tr>
<td>4.21</td>
<td>ISTG KBAT</td>
</tr>
<tr>
<td>4.22</td>
<td>Induced Current Flow for Surface Electro-magnetics (adapted from McNeill, 1980)</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>4.23</td>
<td>Measured Conductivity versus True Conductivity for EM31 (adapted from McNeill, 1980)</td>
</tr>
<tr>
<td>5.1</td>
<td>Tailings Database Site Locations</td>
</tr>
<tr>
<td>6.1</td>
<td>Traditional CPT Soil Behaviour Interpretation Chart (adapted from Robertson and Campanella, 1983)</td>
</tr>
<tr>
<td>6.2</td>
<td>Multi-Zone Soil Behaviour Interpretation Chart (adapted from Campanella and Robertson, 1984)</td>
</tr>
<tr>
<td>6.3</td>
<td>Relative Density Relationships for the CPT (adapted from Robertson and Campanella, 1983)</td>
</tr>
<tr>
<td>6.4</td>
<td>Normalized Piezocone Interpretation Chart (after Robertson, 1990)</td>
</tr>
<tr>
<td>6.5</td>
<td>Unified Piezocone Soil Behaviour Chart</td>
</tr>
<tr>
<td>6.6</td>
<td>Unified Piezocone Chart with Concentric Zones (adapted from Jefferies and Davies, 1993)</td>
</tr>
<tr>
<td>6.7</td>
<td>Ic from ENDK9622 (Endako Pond No. 2)</td>
</tr>
<tr>
<td>6.8</td>
<td>Ic from ENDK9619 (Endako Pond No. 2)</td>
</tr>
<tr>
<td>6.9</td>
<td>Ic from ENDK9603 (Endako Pond No. 1)</td>
</tr>
<tr>
<td>6.10</td>
<td>Ic from BG-T9409 (Gibraltar Pond/Dam)</td>
</tr>
<tr>
<td>6.11</td>
<td>Copper Tailings Tip Resistance versus Gradation (adapted from Steedman, 1997)</td>
</tr>
<tr>
<td>6.12</td>
<td>Normalized CPT Data from Copper Tailings (adapted from Steedman, 1997)</td>
</tr>
<tr>
<td>6.13</td>
<td>Estimating Fines Content from Friction Ratio (%) (adapted from Suzuki et al. 1995)</td>
</tr>
<tr>
<td>6.14</td>
<td>Estimating Fines Content from Ic - Power Relationship</td>
</tr>
<tr>
<td>6.15</td>
<td>Estimating Fines content from Ic - Linear Relationship</td>
</tr>
<tr>
<td>7.1</td>
<td>Simplified Two-Dimensional Soil State Diagram</td>
</tr>
<tr>
<td>7.2</td>
<td>Simplified Three-Dimensional Soil State Diagram</td>
</tr>
<tr>
<td>7.3</td>
<td>Relationship between Critical State Parameters and Material Compressibility</td>
</tr>
<tr>
<td>7.4</td>
<td>Proposed Relationship between F and Material Compressibility</td>
</tr>
<tr>
<td>7.5</td>
<td>Piezocone State Screening Approach</td>
</tr>
<tr>
<td>7.6</td>
<td>Piezocone State for NC Soils - Check of Construct</td>
</tr>
<tr>
<td>7.7</td>
<td>Proposed Piezocone State Screening Compared to SPT (N160-ECS)</td>
</tr>
<tr>
<td>7.8</td>
<td>State Screening for Suncor Tailings Piezocone Data near Liquefaction Slump</td>
</tr>
</tbody>
</table>
Figure 7.9 State Screening for Loose Zone of Sullivan Tailings near Liquefaction Slump
Figure 7.10 Piezocone Screening Applied to Hydraulic Tailings Beach Piezocone Data
Figure 7.11 Comparison between $\psi$ from Proposed Piezocone Approach and $\psi$ from CANLEX Frozen Samples (adapted from Robertson and Wride, 1997)
Figure 7.12 Unification of State Concept
Figure 7.13 Material Compressibility from Seismic Piezocone (adapted from Robertson and Fear, 1995)
Figure 8.1 Summary of Traditional SPT - Piezocone Database (adapted from Burland and Burbidge, 1985)
Figure 8.2 Relationship between Piezocone Material Index and $q_c/N_{60}$ (adapted from Jefferies and Davies, 1993)
Figure 8.3 Measured versus Predicted $N_{60}$ Values - SCL MLSB
Figure 8.4 Measured versus Predicted $N_{60}$ Values - Endako Pond No. 1
Figure 8.5 Measured versus Predicted $N_{60}$ Values - Sullivan Active Iron Pond
Figure 8.6 Measured versus Predicted $N_{60}$ Values - Sullivan Active Iron Pond
Figure 8.7 Measured versus Predicted $N_{50}$ Values - SCL MLSB
Figure 8.8 Measured versus Predicted $N_{60}$ Values - Endako Pond No. 1
Figure 8.9 Measured versus Predicted $N_{60}$ Values - Sullivan Active Iron Pond
Figure 8.10 Measured versus Predicted $N_{60}$ Values - Sullivan Active Iron Pond
Figure 8.11 Measured versus Predicted $N_{60}$ Values - Sullivan Active Iron Pond
Figure 8.12 Sample Empirical Screening Tool for Site Specific Liquefaction Susceptibility (adapted from Conlin, 1987)
Figure 8.13 Simplified Tailings Impoundment Stress States
Figure 8.14 Tailings Impoundment Slopes in Relation to Example Phase Transformation and Anisotropic Collapse Surfaces
Figure 8.15 SPT Liquefaction Susceptibility Chart (adapted from Fear and McRoberts, 1995)
Figure 8.16 Shear Wave Velocity versus Void Ratio - Syncrude Tailings (adapted from Cunning et al, 1995)
Figure 8.17 Normalized Shear Wave Velocity versus Cone Tip Stress - Lower Mainland Deltaic Data (adapted from Cunning et al, 1995)
Figure 9.1 Shear Wave Velocities - Several Mine Tailings
Figure 9.2 Stress Normalized Shear Wave Velocities - Several Mine Tailings
Figure 9.3 Simplified Characteristic Soil Strain Responses to Shear Stress Loading (adapted from Robertson and Fear, 1995)
Figure 9.4 Sample Empirical Screening Tool for Site Specific Liquefaction Susceptibility (adapted from Conlin, 1987)
Figure 9.5 Simplified Tailings Impoundment Stress States
Figure 9.6 Tailings Impoundment Slopes in Relation to Example Phase Transformation and Anisotropic Collapse Surfaces
Figure 9.7 SPT Liquefaction Susceptibility Chart (adapted from Fear and McRoberts, 1995)
Figure 9.8 Shear Wave Velocity versus Void Ratio - Syncrude Tailings (adapted from Cunning et al, 1995)
Figure 9.9 Normalized Shear Wave Velocity versus Cone Tip Stress - Lower Mainland Deltaic Data (adapted from Cunning et al, 1995)
Figure 9.10 Shear Wave Velocity profile at Sullivan Mine Liquefaction Area versus Published Neutral-State Relationships
Figure 9.11 Schematic of Influence Cementation Bonds in Tailings have on In-Situ Characteristics
Figure 9.12 Comparison of Original State Parameter and Integrated Piezocone Approaches - Beaufort Sea Hydraulic Fill Data
Figure 11.10 Sulphate Anion Concentration versus Bulk Resistivity - Tailings Database .......................................................... 350
Figure 11.11 Zinc Concentration versus Bulk Conductivity - Tailings Database ................................................................. 352
Figure 11.12 Copper Concentration versus Bulk Conductivity - Tailings Database .............................................................. 354
Figure 11.13 Magnesium Concentration versus Bulk Conductivity - Tailings Database .......................................................... 355
Figure 11.14 Iron Concentration versus Bulk Conductivity - Tailings Database ................................................................. 356
Figure 11.15 Nickel Concentration versus Bulk Conductivity - Tailings Database ............................................................... 358
Figure 11.16 Cadmium Concentration versus Bulk Conductivity - Tailings Database ........................................................... 359
Figure 11.17 pH versus Bulk Conductivity - Tailings Database ............................................................................................... 360
Figure 11.18 Sample Results from Proposed Resistivity Piezocone Normalization Procedure .................................................. 363
Figure 12.1 Composite Resistivity Piezocone Sounding - Calcine Tailings Area of Sullivan Mine ............................................... 367
Figure 12.2 Composite EM-31 Surface - Calcine Tailings Area of Sullivan Mine ................................................................. 368

LIST OF SYMBOLS

In approximate order of first occurrence:

Φ = objective function for risk-cost-benefit
B(t) = benefits over time
C(t) = costs over time
R(t) = risks over time
γ = utility (in context of risk-cost-benefit)
K = hydraulic conductivity
q_c = cone penetration resistance
G_{max} = shear modulus at small strains
ρ = mass density (in context of stress/strain)
V_s = shear wave velocity
\rho = mass density (in context of electromagnetics)
\sigma = normal stress (in context of stress/strain)
\sigma = conductivity (in context of electromagnetics)
R = bulk resistance
G = bulk conductance
V = voltage
I = current
CF = calibration factor
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega \cdot m$</td>
<td>ohm-metre</td>
</tr>
<tr>
<td>$\rho_b$</td>
<td>bulk resistivity</td>
</tr>
<tr>
<td>$\rho_r$</td>
<td>fluid resistivity</td>
</tr>
<tr>
<td>$S$</td>
<td>Siemens (unit of conductivity in context of electromagnetics)</td>
</tr>
<tr>
<td>$IP$</td>
<td>induced polarization</td>
</tr>
<tr>
<td>$M$</td>
<td>chargeability</td>
</tr>
<tr>
<td>$\delta$</td>
<td>frictional resistance soil-steel</td>
</tr>
<tr>
<td>$D_c$</td>
<td>relative density</td>
</tr>
<tr>
<td>$I_c$</td>
<td>material index</td>
</tr>
<tr>
<td>FC(%)</td>
<td>fines content (&lt; 74 \mu m) in %</td>
</tr>
<tr>
<td>$\psi$</td>
<td>state parameter</td>
</tr>
<tr>
<td>$c$</td>
<td>void ratio</td>
</tr>
<tr>
<td>$e_c$</td>
<td>critical void ratio</td>
</tr>
<tr>
<td>$f_s$</td>
<td>cone sleeve friction</td>
</tr>
<tr>
<td>$F$</td>
<td>stress normalized friction ratio (in context of piezocone data)</td>
</tr>
<tr>
<td>$q_t$</td>
<td>cone penetration resistance corrected for unequal end area</td>
</tr>
<tr>
<td>$Q$</td>
<td>stress normalized cone penetration resistance</td>
</tr>
<tr>
<td>$u$</td>
<td>dynamic cone pore pressure</td>
</tr>
<tr>
<td>$u_0$</td>
<td>static in situ pore pressure</td>
</tr>
<tr>
<td>$B_d$</td>
<td>stress normalized cone pore pressure parameter</td>
</tr>
<tr>
<td>$r$</td>
<td>spacing between critical state line and normal consolidation line</td>
</tr>
<tr>
<td>$R$</td>
<td>isotropic overconsolidation ratio</td>
</tr>
<tr>
<td>$\Lambda$</td>
<td>critical state plastic hardening ratio</td>
</tr>
<tr>
<td>$M$</td>
<td>slope of critical state line in terms of mean effective stresses</td>
</tr>
<tr>
<td>$\phi_c$</td>
<td>angle of shearing resistance at critical state</td>
</tr>
<tr>
<td>$N_{60}$</td>
<td>standard penetration test blowcount normalized to 60% hammer energy</td>
</tr>
<tr>
<td>$(N_1)_{60}$</td>
<td>$N_{60}$ stress normalized to vertical effective stress of 1 bar</td>
</tr>
<tr>
<td>$(N_1)_{60-ECS}$</td>
<td>$(N_1)_{60}$ corrected for fines content to equivalent clean sand value</td>
</tr>
<tr>
<td>$\Delta N_{corr}$</td>
<td>$(N_1)_{60}$ correction for fines content</td>
</tr>
<tr>
<td>$k, m$</td>
<td>constants</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>soil compressibility</td>
</tr>
<tr>
<td>$s_u$</td>
<td>undrained strength</td>
</tr>
<tr>
<td>$D_s$</td>
<td>damping ratio</td>
</tr>
<tr>
<td>CSR</td>
<td>cyclic stress ratio</td>
</tr>
<tr>
<td>CRR</td>
<td>cyclic resistance ratio</td>
</tr>
<tr>
<td>$r_d$</td>
<td>empirical stress reduction factor</td>
</tr>
<tr>
<td>$S$</td>
<td>slope of tailings embankment</td>
</tr>
<tr>
<td>$H$</td>
<td>height of tailings embankment</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>unit weight of tailings</td>
</tr>
<tr>
<td>$N_s$</td>
<td>stability factor</td>
</tr>
<tr>
<td>$c_{v_h}$</td>
<td>coefficients of consolidation</td>
</tr>
<tr>
<td>$m_v$</td>
<td>volumetric compressibility</td>
</tr>
<tr>
<td>$T$</td>
<td>time factors of consolidation</td>
</tr>
<tr>
<td>$F$</td>
<td>flow factor (in context of hydraulic measurements)</td>
</tr>
<tr>
<td>$A$</td>
<td>slope of total pressure head (dissipated values)</td>
</tr>
<tr>
<td>$B$</td>
<td>slope of phreatic surface</td>
</tr>
</tbody>
</table>
\[ v_{x,y} = \text{Darcian flow velocity} \]
\[ h = \text{hydraulic head} \]
\[ F = \text{formation factor (in context of bulk resistivity)} \]
\[ n = \text{porosity} \]
\[ S_r = \text{degree of saturation} \]
\[ m = \text{cementation factor} \]
\[ s = \text{saturation exponent} \]

ACKNOWLEDGMENTS

There are a significant number of individuals, corporate entities and government agencies whose assistance made this research possible. Although the following is lengthy, the Author’s sincere appreciation would not be complete without listing the key contributors.

First and foremost, the support of my family, Carolyn my partner, and our two children, Taylor and Zachary, has been instrumental in allowing me to carry out my doctoral studies.

The Author received financial support from the Science Council of British Columbia (SCBC) through their innovative and generous STARS program for industry-mature researchers. The Author also received an academic research scholarship from the Natural Sciences and Engineering Research Council of Canada.

The mentorship and support provided by Dr. Richard G. Campanella was tremendous. His quick and constructively critical mind and enthusiasm for field research have greatly influenced this research and my career in general. It was an honour to work with “Campy” on my doctoral research.

The various fellow students who worked with me in the In-Situ Testing Group (ISTG) over the five years of my research number greater than 20. All of you are appreciated. Particular thanks to Messrs. Tim Boyd and Scott Tomlinson and Dr. Debasis Roy for their field assistance and spirited technical discussions.

The skills of the technician support available to the ISTG was essential to the tool development and data acquisition software central to my research. To Messrs. Scott Jackson, Harald Schrempp, Kent Dang and Thomas Wong; heartfelt thanks.

For the SCBC project, really the key element to the data obtained for my thesis, the funding support of the SCBC, Placer Dome Inc. and Cominco Ltd. is gratefully acknowledged. Particular thanks to Mr. Jim Robertson of Placer and Mr. Walter Kuit of Cominco. The equipment transportation costs, chemical laboratory testing costs, all travel and accommodation expenses and technical costs were covered by these funding sources. Specific thanks to Sullivan Mine, and in particular Messrs. Bruce Dawson, Gray Gibson and Bob Gardiner, Gibraltar Mine and in particular Messrs. Bob Patterson and Todd Wamboldt, Endako Mine and in particular Messrs. George Paspalas and Bill Price and the Trail Smelter Site and in particular Mr. Bob
Abbey. All of the mine sites visited were generous with their time, staff, equipment (e.g., creating vehicle access) and fuel for our research vehicle.

For the CANLEX work, various students from the Universities of British Columbia and Alberta provided their assistance. Financial support and equipment provided by Syncrude Canada Ltd. and the British Columbia Ministry of Transportation and Highways is acknowledged. Particular thanks to Mr. Beat List of Syncrude Canada Ltd.

The MENDO-INCO project received its direct financial support from the Ontario Ministry of Northern Development and Mining, through its MEND (Mine Environment Neutral Drainage) Ontario initiative, and the Ontario and Manitoba Operations of INCO Ltd. This financial support is gratefully acknowledged and was mainly used to provide support for technician staff, equipment shipping, travel and the field program. INCO Ltd. further provided in-kind support of site access, personnel assistance and other similar items which made the project both possible and successful. Particular individual thanks to INCO personnel Messrs. Marty Puro, Rodney Stuparyk, and Dr. Bill Kipkie whose collective support for this demonstration project resulted in its reality. Support from CANMET by way of introducing the concept to MEND is appreciated; Ms. Marcia Blanchette and Mr. Grant Feasby assisted in formulating the proposed work package and fitting it into the 1993 field season. Dr. David Blowes and his colleagues at the University of Waterloo provided field geochemical data. Falconbridge Ltd., and in particular Mr. Mark Wiseman, is also acknowledged for providing site access.

The assistance of Klohn-Crippen Ltd., the Author’s employer during the research tenure, was important to the success of this thesis. AGRA Earth & Environmental, the Author’s present employer, have been equally supportive. Klohn-Crippen’s Sudbury office provided office space and technical support during the field portion of the MENDO-INCO project. Klohn-Crippen’s main office in Richmond, British Columbia provided significant support to the various projects with no-cost accounting, report collation and drafting services to both the MENDO-INCO and SCBC projects. Furthermore, the typographical skills of Melissa Neuman contributed greatly to the research documentation.

The ISTG of the University of British Columbia operates from funds raised by applied research programs, industrial grants and Federal and Provincial Government support. All Canadian taxpayers, most of which will never read this work that they have indirectly funded, are gratefully acknowledged as the specific means by which the Author was able to fund his research tenure and the general means by which Universities can bring such applied research initiatives to industry.
1. **INTRODUCTION**

1.1 **General**

The storage of mine tailings represents perhaps the single largest environmental liability to a mining development. Mine tailings storage facilities, whether traditional impoundments, dewatered "stacked" deposits or something in between, include several of the world's largest manmade structures. Although the majority of the world's tailings facilities are in the modest size range of between 20,000 m$^3$ to 100,000 m$^3$, these facilities can all usually claim relative large size as one of their defining attributes when compared to most civil earthwork entities. There are some dramatic examples of the enormity that mine tailings storage facilities can attain. For example, some of the Author's site characterization experience has included INCO Canada Limited's Copper Cliff tailings storage area near Sudbury, Ontario (~800,000,000 m$^3$ of mine tailings), Highland Valley Copper's tailings impoundment near Merrit, British Columbia (~1,000,000,000 m$^3$), Syncrude Canada Limited's Mildred Lake tailings facility near Fort McMurray, Alberta (~800,000,000 m$^3$), Kennecott Utah Copper's Magna tailings impoundment near Salt Lake City, Utah (~900,000,000 m$^3$) and Syncrude's Southwest Storage Site tailings facility (planned ~1,000,000,000 m$^3$).

There have been roughly two major tailings impoundment failures per year since the 1950's. Many of these failures have been directly attributed to a lack of understanding of the in-situ condition of the tailings, both geotechnically and geochemically. It would therefore appear that either appropriate methods for tailings characterization have not been available or the available methods have not been utilized for whatever reasons. From the Author's experience, the lack of
appropriate characterization technology (technically and/or economically) has contributed to the insufficient characterization of many tailings impoundments.

With essentially all current tailings facilities of interest being of considerable size, optimal characterization for engineering evaluation requires utilization of both a rational investigation framework and the "best available" technology for any given parameter assessment. Sampling a statistically representative volume is clearly not an option. To be effective (pragmatic), "best available" must be a balance between benchmark technology and the cost per unit of facility characterized by the utilization of a given technology. As far as investigative frameworks, such a choice should embody the concept of value of information. Each investigation location usually represents a significant real cost and the data obtained is the only real approximation of true in-situ character.

For best available technology, there are several attributes that should be met. Ideally, it should provide continuous data of high quality, covering as great a site volume as possible, and provide data that is obtained at prevailing in-situ geomechanics and chemical state conditions. Such attributes should be viewed as the benchmark. Due to the typical large size of tailings facilities involving considerable spatial variability in physical and chemical properties, traditional discrete physical sampling and subsequent laboratory testing is not necessarily a best available technology. The traditional discrete sampling approach, when used as the sole site characterization methodology for assessing tailings facilities, will usually be prohibitively expensive if a sufficient quantity/volume of truly representative materials are to be procured and tested. Moreover, the generally cohesionless nature and unique fabric of mine tailings makes
physically representative sampling of these tailings difficult. A characterization approach that allows sufficient spatial coverage, appropriate accuracy of result and overall volumetric cost effectiveness of material tested is ideally required.

1.2 This Research

In-situ testing, and in particular piezocone technology, is often considered a candidate “best available technology” for geotechnical parameter assessment in many soil conditions. The parameters estimated include values for both screening level evaluations as well as for more accurate determinations. The aim of the research described in this thesis has been to demonstrate whether piezocone technology can offer similar advantageous technology to the geoenvironmental characterization of mine tailings.

The research initiative carried out to support the thesis involved:

- using existing industry standard and research level-piezocone technology at a number of mine tailings facilities and calibration research test sites;
- developing new and/or modifying existing piezocone related tools/modules to create optimal systems for the research objectives and potential commercial characterization work;
- evaluating available parameter assessment procedures for soils in general and, where appropriate, recommending the most suitable for mine tailings based upon the research data - this evaluative work includes noting possibly inappropriate piezocone assessment procedures where mine tailings are involved;
- where suitable parameter assessment procedures for mine tailings could not be found from available relationships, development of recommended improvements to these procedures or proposed new procedures was carried out; and
- demonstrating the practicability of both the tools and the selected parameter assessment procedures with substantive examples from actual mine tailings sites.
The piezocone technology used and developed for this research was mainly the resistivity piezocone. Companion discrete-depth water sampling technology, also modified during the tenure of the research, aided immensely in the research. The concept of applying the latest piezocone technology to actual industry situations was a key part of the research. It was felt at the outset that effective development and transfer of the technology to industry would be best achieved by demonstrating its effectiveness in solving practical site characterization problems. As such, a considerable portion of time was spent at mine sites using the technology on various tailings facilities.

The results of the research should also have application to industries outside of mine tailings although the Author suggests caution in applying any of the original relationships without a thorough understanding of their potential limitations in the given application.

1.3 This Thesis

Can piezocone technology contribute to the technically acceptable and economically palatable characterization of mine tailings? This was the central problem statement used by the Author in designing the research program for this thesis and in evaluating the results from a program which spanned six years and involved a wide variety of mine tailings and their unique geoenvironmental characterization issues.

The research described herein demonstrates to what degree existing and newly developed piezocone technology will permit accurate geoenvironmental characterization of the soil-water system within mine tailings storage areas. Geotechnical parameters such as in-situ state, shear
tendencies (contraction versus dilation) and hydrogeologic character form part of this characterization. The geochemical attributes of interest for mine tailings include pH, bulk and pore fluid conductivity, and concentration and type of dissolved complexes of solids and/or dissociated ions. Of special concern to the mining industry are tailings from sulphide-bearing ores. In cases of sulphide-rich tailings, there is the potential oxidation of these materials leading to acid generation and the subsequent potential for high metal release to off-site receiving environments.

The research was carried out over six years and involved more than 120 days of remote field testing and more than 60 days of local calibration test site work. Consequently, the database for the research is relatively large. The research included several large "projects". The three key projects used to support of the research are briefly described in Chapter 5 and were (acronyms defined in Chapter 5 and Appendix I):

1. the SCBC joint industry-government project aimed at evaluating the application of piezocone and near surface geophysical techniques to the geoenvironmental characterization of mine tailings;

2. the MEND-INCO joint industry-government project evaluating the applicability of resistivity piezocone technology to sulphide-rich mine tailings where acid rock drainage was occurring; and

3. the CANLEX initiative, again a joint industry-government effort, where evaluation of the ability of in-situ tools, such as the piezocone, to assess liquefaction phenomena was carried out in mine tailings and deltaic calibrative sites.
The first two projects were conceived and developed by the Author and executed as part of this doctoral research. The latter was conceived and managed by others, though the Author had participation in the acquisition of piezocone data for the CANLEX project.

Several local (to the University of British Columbia) calibration sites were utilized in allowing test locations for new piezocone tool development efforts. Projects in the Lower Mainland for the Geological Survey of Canada and Domtar Ltd. were also useful in assessing the technologies developed for this research under a wide-variety of actual “project” conditions.

In total, there were more than 200 separate piezocone soundings, 100 discrete-depth water samples and 50 km of near-surface electro-magnetic surface geophysical survey “lines” acquired during the research. This database has allowed a broad-range of topics within the general theme of geoenvironmental characterization of mine tailings to be investigated.

From the various sub-projects related to the doctoral research, there were a number of documents produced during the tenure of the research. These are considered to represent very key appendices to this thesis. Being impractical to include even single-page plots of just the piezocone soundings carried out in formulating the assertions in this thesis, these documents are referenced and readers desiring more of the background data used are encouraged to consult these references.

Each chapter of this thesis, save this introduction and Chapters 2 and 5, were developed to be brief, stand-alone documents addressing an aspect of piezocone technology and/or specific mine
tailings characterization topic. Consequently, the chapters do not necessarily build upon one another and can be utilized in isolation as befits the relatively broad range of topics covered.

Most every topic addressed in this thesis deserves more discussion and evaluation than was provided by this dissertation. However, it was a prime objective to create a unique contribution that addressed the key areas to practical geoenvironmental characterization of mine tailings versus one specific aspect of such characterization. This thesis, like the research, is indeed broad-based; minimum goal being an initial document of its kind for both researchers and practitioners dealing with the unique challenges involved with characterizing mine tailings.

1.4 Engineering Disclaimer

This thesis presents data from a number of operating mines and related sites in Canada. The data is presented to support the thesis statement and should not be used by third parties for drawing engineering conclusions regarding any of the tailings facilities for any of the mines. The data presented in this thesis was not obtained as part of focused engineering studies. The indication that, for example, some of the tailings tested exhibit certain geochemical and/or geotechnical character does not imply that this is the case for any appreciable volume of tailings at the given minesite.

Neither the Author nor any given minesite supports any minesite specific conclusions drawn from the data contained in this thesis that is not explicitly stated in the thesis.
Use of any data contained within this thesis to draw engineering conclusions may only be carried out with the expressed written permission of the given minesite and with full knowledge of the Author. Scholarly use of the data and suggested relationships developed from the data is naturally encouraged.
2. GEOENVIRONMENTAL SITE CHARACTERIZATION

2.1 General

In the course of the doctoral studies, the Author became exposed to a wide range of decision-making tools applicable to developing a tenable site characterization program. These few pages present such a framework through key definitions, a recommended characterization philosophy and suggested overall relevance to mine tailings geoenvironmental characterization programs. Inclusion of the philosophy presented herein is purely for the Author’s benefit and can be ignored in any review of this thesis. Nonetheless, any sound site characterization program must be formulated using some logical framework.

2.2 Basic Definitions

Geoenvironmental is a relatively new term in the engineering and geoscience field. Available literature includes a number of interpretations of what the term “geoenvironmental” exactly means. To avoid confusion the definition used herein, and the one seemingly most consistent with original intent, is the following suggested by the Author:

"the field of study that links geological, geotechnical and environmental engineering and their related engineering sciences to form an area of study and practice that includes all physical and geochemical concerns within natural or processed geological media."

Whether carrying out an evaluation of estimated liquefied soil strength or conducting groundwater contamination studies, both are excellent examples of areas of interest within the broad scope of the geoenvironmental field. With the basic definition established, the Author suggests that subsurface geoenvironmental site characterization can then be defined as:
"the procurement, collation and interpretation of available physical and chemical information from the site in question which allows adequate three-dimensional representation of the prevailing subsurface conditions in the context of the engineering requirements of the data."

For many soil conditions, modern in-situ testing instruments are increasingly being used for developing this three-dimensional representation. In-situ testing is carried out under prevailing field conditions and has the potential to show the effect these conditions have on material behaviour. On the other hand, laboratory testing, usually on disturbed samples, attempts to estimate the many factors that make up "field" conditions. However, in-situ tests rarely allow physical or geochemical stress-path control. Consequently, a sensible combination of field screening with in-situ testing followed by select laboratory testing would appear to be a good approach to adequate geoenvironmental site characterization.

With in-situ tests, the value of a given test for geoenvironmental site characterization is directly related to how much non-quantifiable disturbance and/or test-to-test variance in disturbance is caused during tool insertion required to carry out the test; e.g., the degree of data repeatability. These issues are addressed later in the thesis.

2.3 Characterization Philosophy

There has been a trend in site characterization literature to stress the importance of following appropriate geostatistical models for any characterization projects. However, geostatistical methodologies alone without some form of quantifying judgment can show that the sampling needs are so great and involve such large masses of material with such regularity of sampling that the practicality of the exercise is lost. When using geostatistics to guide mine tailings
characterization, a resulting state of non-predictability results because the volume of the system requiring characterizing is so large.

Figure 2.1 is taken from the British Columbia Acid Mine Drainage Task Force Guide (BC AMD Task Force, 1989). Using the recommended approach shown in Figure 2.1, a minimum number of samples (say water samples for water chemistry or soil samples for strength testing) for "average" base-metal tailings facilities (approximately $50 \times 10^6$ to $250 \times 10^6$ tonnes) would be in the order of 50 to 150. For the very largest facilities, more than 250 samples would be suggested. Clearly, this number of samples is extensive and the associated costs considerable. For example, assume that a separate drill hole was required for each sample extracted. There is a very real need to carry out such extensive sampling programs effectively (sample in the locations that provide the most information) and efficiently (try to minimize the required number of samples). It is more common to see a dozen or so samples used for soil strength on a typical tailings facility - far fewer than a statistically representative sampling. What then is the best approach to ensuring effective and efficient site characterization programs?

To help answer this question, the concept of data worth can be utilized. As defined by Freeze et al. (1992), data worth is established by comparing the cost of data collection against the expected value (which can also be considered the risk reduction) that the data provides. From the stated premise, data can only have worth if they aid in making optimal decisions between alternative courses of action; e.g., where to place a piezocone sounding or at what depth to procure a soil sample. These courses of action, for geoenvironmental site characterization, are the judgment that has typically lacked quantification in practice.
Figure 2.1 Recommended Minimum Sampling Quantity versus Size of Unit (adapted from BC Acid Drainage Task Force, 1989)
A full explanation of data worth is beyond the scope of this thesis. Briefly, however, it is based upon the concept that the approach to site characterization can be evaluated in a simple fashion acceptable for stochastic systems; e.g., the geoenvironmental conditions within mine tailings.

The data worth concept can be simplified in terms of an objective function given as:

$$\Phi = \sum_{t=0}^{T} \frac{1}{(1+i)^t} [B(t) - C(t) - R(t)]$$

(2.1)

where

$$R(t) = P_f(t) C_f(t) \gamma(C_f)$$

(2.2)

In equations 2.1 and 2.2, the stream of benefits, B(t), costs, C(t), and risks, R(t) are discounted at a rate, i, and summarized over a reasonable time horizon, T. The benefits arise from the ability to, for example, maintain mining operation whereas the costs include the capital and operating costs of geoenvironmental site characterization programs. The risks are determined by the probability of failure, P_f(t) (e.g., not meeting compliance or missing a weak layer that later leads to impoundment failure), the costs associated with failure, C_f(t), and a utility function \( \gamma(C_f) \) which can be taken to be equal to unity for risk neutral decisions and increases with the adversiveness to risk for each particular case analyzed. An owner of a tailings facility would, for example, be more risk adverse to an impounding dam failure than to marginal effluent non-compliance for some defined period and level of monetary fine. More complete definitions of risk, including practical tools for qualitative risk assessments, are provided in the Author’s paper, Davies (1997).
How data worth is actually calculated presents a real attraction to planning practical site characterization programs. In equations 2.1 and 2.2 there are terms that require input from technical consultants, regulators and, most importantly, owners of the tailings facilities. Having owners involved in the decision making process about how much geoenvironmental site characterization is required to maintain the benefit of the tailings facility will make implementing appropriate programs a much more straightforward task with much of the mystery removed. Past reluctance on the behalf of owners to allot necessary resources to site characterization was likely rooted in the non-consistent nature of recommended approaches and the lack of ability to quantify what such programs meant to the overall mining operation.

Data worth as defined by hydrogeological literature uses a Bayesian approach by evaluating the objective functions at prior, posterior and preposterior stages. Alternatives compared before a site characterization program represent the prior analyses. Alternatives compared after the characterization program is completed represent posterior analyses. The preposterior analyses is an intermediate step and one that is carried out after specifying the number and location of proposed measurement points for the proposed site characterization program but before any actual data collection has occurred. By comparing the difference in the objective functions between the prior and preposterior analyses, which will be different, an actual quantity is available. This quantity, in monetary units, is the data worth for that particular planned site characterization program.

Data worth can probably best be used in mine tailings characterization programs by assisting in the determination of how much should (or should not) be spent to reduce geoenvironmental
uncertainty by a given amount. In other words, by using simple or sophisticated search techniques and statistical models, the cost to get a program to the point where, for example, the copper loading in the groundwater system will be known to +/- 5 mg/litre in the aquifer being characterized, can be computed. If the cost of achieving this level of site characterization far exceeds or, more commonly, even approaches the cost of a known effective proactive or remedial action, the data worth will be very small or even negative and the proposed characterization program would be a wasted expense.

Another important use of data worth is to assist proponents of characterization programs in demonstrating to those ultimately bearing the costs of such programs that such costs are proactive and can save many times the outlay over time. The storage of mine tailings represent placement and stewardship of materials that do not make money for the producer but only add cost to the mineral extraction process. The reluctance to allot resources to these waste materials is fully understandable and, hence, having a tool like data worth to demonstrate to producers, using their own input values, that reasonable resource application is prudent can help get appropriate characterization programs initiated or to maintain needed programs over extended periods.

Whether the above form of data worth is adopted by all who characterize mine tailings sites is not as important as that some form of judgment quantification be used. The method used should be repeatable and based upon some rational decision process that resists becoming overly biased. The real aim of the quantification is to try and keep the site characterization at the “forest” level versus the “tree” level; e.g., avoid having the site characterization overly focused on one detailed aspect that uses most of the available resources when a less costly, pragmatic solution may be
available. Additionally, prejudging physical and chemical systems for items such as, for example, groundwater flow direction or foundation strength distribution will lead to possibly ignoring statistically small, but actually of prime importance, data trends that are not pursued because the data worth of such a pursuit is not clear. For example, the best sample location is of the point of greatest uncertainty which, as luck often has it, is not at one of the four corners of the site and/or not where is of most convenience to the site investigation team.

2.4 Requirements for Mine Tailings Projects

As described in Chapter 3, mine tailings represent a relatively wide-range of materials. Within this range of materials are a number of key physical and chemical properties. To assess these properties, the typical requirements for appropriate geoenvironmental characterization includes:

- a profile of soil behaviour units (often very similar to the stratigraphic profile);
- density and stress states for each behaviour type;
- relative geotechnical properties of each soil behaviour unit under the governing density state and stresses;
- hydrogeological properties of each soil behaviour unit; and
- the nature of the pore fluids that are present in each behaviour unit.

The assessed value of any site characterization tool or technique for mine tailings must include an evaluation of whether it can provide one or more of the above requisite items and how comprehensive and accurate the results provided will be. The value of the technology assessed in this research is judged in comparison to all of the above requirements.
3. **MINE TAILINGS**

3.1 **Overview**

This chapter of the thesis is largely from another publication by the Author developed during the research tenure. While completing this research, the British Columbia Ministry of Energy, Mines and Petroleum Resources (then Ministry of Employment and Investment, Mines Inspectorate), retained the Author to develop British Columbia’s Tailings Dam Inspection manual (Davies, 1996). Copies of this manual are available from the Government of British Columbia.

3.2 **General**

To adequately characterize any soil deposit, a thorough understanding of deposit genesis and typical material properties for similar deposits is an essential first step. Therefore, for this research it was important to examine how mine tailings are formed and what they typically represent once deposited.

Mine tailings are unique geologic materials in that, although natural in their mineral composition, they are physically and chemically processed and do not readily find equivalency in natural deposits. It is not uncommon to have a given tailings with more than 70% to 90% clay-sized material having the constitutive behaviour of a clean sand. This same material may consist entirely of angular and sub-angular grains yet reside in thinly laminated deposits consistent with high-energy fluvial environments. Moreover, from a stress-history perspective, mine tailings can be placed within storage facilities at rates of several metres per year that is beyond most of the experience base with natural deposits. Mine tailings deposits are also geologically “young” and can undergo extensive aging related changes in character over their operating life.
A synoptic summary of where the majority of mine tailings are derived is presented along with the nature of typical deposition structures and material characteristics. The overview provides terminology useful later in the thesis to those not as familiar with mine tailings and/or the common modes of surface storage of these materials.

3.3 Historical Perspective

Mining has been carried out in some form for at least 5000 years. It would appear that each major civilization following roughly 3000 BC owed much of its advancement to utilizing materials from some form of mining process. Larger scale efforts came following the Dark Ages. Crude millstone crushing of grinding ore was initially practiced in the 1500s and continued through the mid-1800s. The largest change over that 3 to 4 centuries was the introduction of steam power which greatly increased the capacity of the grinding mills and hence, the amount of non-ore byproduct. Ore minerals were separated from the crushed rock according to differences in specific gravity. The remaining "barren" particles, or tailings, were traditionally routed to some convenient location. More often than not, the location of greatest convenience was the nearest stream or river where the tailings were then removed from the deposition area by base flows or larger infrequent storm events. Tailings storage concerns were largely eliminated with little effort.

Later in the 1800s, two significant developments occurred which revolutionized mining:

1. the development of froth flotation; and
2. the introduction of cyanide for gold extraction.
Flotation resulted in the production of still larger quantities of tailings with even finer gradation (e.g., more material in the < 74 μm range) and, as per cyanidation, greatly increased the world’s ability to mine less-rich ore bodies. However, tailings disposal practices remained largely unchanged and, as a result, more tailings were being placed into streams and rivers and being subsequently transported over greater distances into receiving waters.

At around 1900, remote mining districts of appreciable size began to develop and these developments began to attract supporting industries and agricultural development. Conflicts soon developed over land and water use, particularly where agricultural interests were involved. Accumulated tailings regularly plugged irrigation ditches and “contaminated” downstream growing areas. Farmers began to notice lesser crop yields from tailings influenced lands. In extreme cases, livestock became ill or died from drinking water effected by mine tailings. Issues with land and water use that formed the basis for these initial conflicts led to the first mine tailings litigation in both North America and Europe. Legal precedence gradually brought an end to uncontrolled disposal of tailings in most of the “western world” with a complete cessation of such practices occurring by about 1930. To retain the ability to mine and profit, industry fostered construction of some of the first dams to retain tailings. Many of these early dams served both to capture tailings in streams and to store water for mill process use during dry periods. However, the dams were often built across the stream channel in such a manner to provide only limited provisions for passing statistically infrequent floods. Consequently, as larger rainfall events or more intense freshet periods occurred, few of these “in-stream” dams survived. Very little, if any, engineering or regulatory input was involved in the construction or operation of these early tailings dams.
Mechanized earth-moving equipment was not available to the early dam builders. As a result, a hand-labor construction procedure (the initial upstream method) was developed whereby a low dyked impoundment was initially filled with hydraulically-deposited tailings, then incrementally raised by constructing low berms above and behind the dyke of the previous level (Vick, 1994). This construction procedure, now almost always mechanized, remains in use at many mines even today.

The first departure from the above procedure likely resulted from the failure of the Barahona tailings dam in Chile. During a large earthquake in 1928, the Barahona dam failed killing more than 50 people. The Barahona dam was replaced by a more stable downstream dam that used cyclones to procure coarser-sized material for dam construction from the overall tailings stream. By about the 1940’s, the availability of high-capacity earth-moving equipment, especially at open-pit mines, made it possible to construct tailings dams of compacted tailings and/or borrowed earthfill in a manner similar to conventional water dam construction practice and with a corresponding higher degree of safety (Vick, 1994).

The development of tailings dam technology proceeded on a purely empirical basis geared largely to the construction practices and equipment available at the time. This development was largely without the benefit of engineering design in the contemporary sense. Nonetheless, by the 1950’s many fundamental dam engineering principles were understood and applied to tailings dams at a number of mines in North America. It was not until the 1960’s, however, that geotechnical engineering and related disciplines adopted, refined, and widely applied these empirical design rules. The 1965 earthquake-induced failures of a number of tailings dams in
Chile received considerable attention and proved to be a key factor in early research into the phenomenon of tailings liquefaction. Liquefaction issues remain key considerations in tailings dam design throughout the world.

Issues related to the environmental impacts from tailings dams were first seriously introduced to the engineering process in the 1970's in relation to uranium tailings. However, environmental issues related to mining received attention for centuries. For example, Agricola (1556) in the 16th century stated:

"The strongest argument of the detractors of mining is that the fields are devastated by mining operations... Further, when the ores are washed, the water which has been used poisons the brooks and streams, and either destroys the fish or drives them away... Thus it is said, it is clear to all that there is greater detriment from mining than the values of the metals which the mining produces."

The above was written almost 450 years ago yet the Author has met nearly identical rhetoric throughout the world during his own career.

As another example of this attention, public concerns about the effects of acid rock drainage have existed for roughly 1,000 years in Norway. Acid rock drainage issues are still topical today.

Environmental issues are growing in importance as public attention has largely turned from mine economics and physical stability of tailings dams to their potential chemical effects and contaminant transport mechanisms. Recent catastrophic physical failures such as Stava, Italy in 1985, Merriespruit, South Africa in 1994, Omai, Guyana in 1995 and Los Frailes, Spain in 1998 illustrate this issue with most of the media reports highlighting the human and environmental
impacts of the failures. There was scarcely any mention as to the economic impact to the mining company or the direct and indirect labour force brought about by the failures. One unfortunate footnote to the latter three failures noted is that these facilities were designed and constructed within the past decade using apparently “state-of-practice” techniques.

3.4 Tailings Characteristics

3.4.1 Types of Tailings

Except in cases such as oil sands mining, mine tailings consist of the ground-up rock from milling processes that remains after practical economic value has been removed from the ore. With oil sands and mineral sands, the tailings consist of the natural soil that remains after the economic product has been removed without crushing processes and tends to retain the grain-shape of the in-situ ore deposit.

The textural (grain size) distribution of tailings depends upon the characteristics of the ore and the mill processes used to concentrate and extract the metal values. A wide range of tailings gradation distribution exist for various mining operations. Depending on the ore and milling techniques, tailings range from predominantly coarse-sand to predominantly clay-sized particles with all variety of mixtures in between. Figure 3.1 presents some typical gradations for different
Figure 3.1   Typical Tailings Gradations (adapted from Klohn, 1995)
types of tailings. For most metal mines 40% to 70% of the tailings will pass through an equivalent No. 200 sieve (0.074 mm). However, some milling processes such as gold extraction may grind the ore so that 90% or more of the tailings pass the No. 200 sieve.

3.4.2 Depositional Characteristics

Tailings are almost always transported from the mill to the deposition area in a pipeline as a slurry of water and solids. The slurry typically exists at pulp densities ranging from about 25% to 55% solids by weight. This means that for every tonne of tailings deposited, somewhere between one and three tonnes of water is also deposited. The amount of water present is the main controlling variable on the depositional characteristics of the tailings. The amount of free water that is impounded in the tailings storage area varies from very small at mining operations where most of the water is decanted and returned to the mill, to very large for disposal areas that are used for water storage as well as tailings storage. Climatic conditions (e.g. difference between precipitative and evaporative potentials) are also a controlling factor. The volume of free water that is stored with the tailings has a fundamental control on what constitutes appropriate design and operations of a given tailings storage facility.

In surface impoundments, the slurry is most often discharged from the crest of the dam. This occurs either through spigots in the tailings pipeline, typically spaced 10 m to 50 m apart, or by open end single-point discharge. As the solids settle from the discharged slurry (subaerial), a gently-sloping beach above-water (BAW) forms starting at the discharge point. This BAW zone extends out to the decant pond where water remaining from the slurry and all natural precipitation and runoff accumulates. Tailings that become deposited below the pond surface (subaqueous) are termed as part of the beach below-water (BBW) zone.
In theory, coarser particles in the tailings stream settle out first from the slurry, finer particles further out on the beach, and the finest silt and clay-sized particles settle from suspension in the decant pond. The actual degree of particle size segregation varies greatly both within a given deposit and from one deposit to another. This variation is due to such factors as the gradation of the mill grind, solids content, climatic conditions and flow rate of the discharged slurry. These same factors influence the beach geometry(s) and in-place density of the beach tailings.

Beach slope formation is essentially analogous to natural prograding deltaic deposition processes. For “mature” tailings impoundments, e.g., where beaches have been established, the BAW slope will decrease with distance from the point of tailings deposition. The type of tailings is important in what slope is established but there are few tailings beaches which can be modeled after another. The empirical database can be used (e.g., Conlin, 1989) to group some tailings types but it is difficult to draw repeatable conclusions. For general “rule-of-thumb” guidelines, coarse tailings (defined below) can have BAW beaches of 1.5% to 4% over several hundred metres. Fine tailings (defined below) slopes are flatter. BBW slopes are steeper, up to double BAW slopes, at the same distance from discharge. Reagents, such as flocculants, tend to occasionally affect BBW slopes but rarely affect BAW slopes. After several hundred metres, and often less distance, all beach slopes tend to become 0.2% to 0.5% regardless of tailings grind or deposition method.

Segregation creates zones of tailings within the deposit that can be classified by grain size. Current mining terminology has two subsets within this gradation classification:
• **Coarse** tailings are materials predominantly coarser than 0.074 mm, i.e., with less than 50% finer than this size.

• **Fine** tailings are predominantly silt-sized material with more than 50% finer than 0.074 mm.

Old terminology referred to coarse tailings as “sands” and fine tailings as “slimes”; these terms are neither currently common nor desirable.

In most operating impoundments, the size and location of the decant pond varies over time. Spigotting methods and discharge locations also change and the coarse and the intermediate coarse-fine transition zones may be poorly developed or differentiated. As a result, highly heterogeneous deposits of horizontally and discontinuously layered coarse and fine tailings often develop in tailings facilities.

### 3.4.3 Typical Physical Properties

The physical characteristics of tailings deposits results from the nature of their parent rock, the milling process and the form of their hydraulic deposition. Understanding these characteristics is important in understanding how the deposit will respond to static loading, seepage forces, and transient loads such as seismic shaking. Familiarity with typical values can assist when carrying out site-specific characterization work. Examples of typical values for frictional resistance and bulk hydraulic conductivity for some common tailings are presented in Table 3.1.
Table 3.1  Typical Physical Parameters of Various Mine Tailings

<table>
<thead>
<tr>
<th>TAILINGS MATERIAL</th>
<th>PEAK FRICTIONAL RESISTANCE $\phi$ (Drained)</th>
<th>BULK HYDRAULIC CONDUCTIVITY $K$, cm/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Copper cycloned sand - coarse</td>
<td>34°</td>
<td>$10^{-2}$ to $10^{-3}$</td>
</tr>
<tr>
<td>Copper cycloned sand - fine</td>
<td>32°</td>
<td>$10^{-4}$ to $10^{-8}$</td>
</tr>
<tr>
<td>Molybdenum beach sands</td>
<td>35°</td>
<td>$10^{-4}$ to $10^{-5}$</td>
</tr>
<tr>
<td>Lead-zinc coarse tailings</td>
<td>34°</td>
<td>$10^{-5}$ to $10^{-6}$</td>
</tr>
<tr>
<td>Lead-zinc fine tailings</td>
<td>30°</td>
<td>$10^{-6}$ to $10^{-8}$</td>
</tr>
<tr>
<td>Gold tailings</td>
<td>28°*</td>
<td>$10^{-6}$ to $10^{-7}$</td>
</tr>
<tr>
<td>Oil sand tailings</td>
<td>36°</td>
<td>$10^{-2}$ to $10^{-3}$</td>
</tr>
</tbody>
</table>

*can be higher

3.4.4 Typical Geochemistry

Likely the most typical geochemical characteristic of tailings storage facilities is that there is no typical character. The actual mineralogy of the orebody, the milling methodology, the nature of the storage facility, site hydrology and other factors combine to make each tailings facility geochemically unique. When characterizing a facility, the three most fundamental pieces of information required are:

- chemistry of tailings slurry water;
- tailings mineralogy; and
- tailings storage facility water balance.

For example, knowing the pH of the tailings stream, lack or presence of sulphide minerals and estimated dilution from runoff would be important components to an adequate characterization effort.
3.5 Tailings Storage Options

3.5.1 General

Surface tailings impoundments are traditionally the most versatile and economic tailings disposal option for most mining operations. This option relies on hydraulic deposition of tailings behind a dam(s) that can be constructed using a variety of materials and configurations. The dam is intended to confine the tailings slurry to:

- allow the solids to settle; and
- recirculate process and hydrological water to the mill.

A comparative review of tailings disposal practices was published previously by the Author (McLeod et al., 1992).

3.5.2 Types of Tailings Dams

Although there are several older examples, starting around 1960, larger open-pit mining operations have become more common. These operations, which have been made possible in part by the development of very large and efficient earth and rock moving equipment, produce huge volumes of tailings. Open pit metal mines producing 100,000 to 250,000 tonnes per day of tailings are not uncommon. The largest open pit mining operations are in the oil sands operations in Alberta and the base metal mines in South America. For example, approximately 250,000 tonnes per day of tailings are produced at the Syncrude oil sands operation alone. For comparison, British Columbia’s largest volume tailings producer, Highland Valley Copper, produces roughly 130,000 tonnes per day.
To meet the storage demands of these operations, many tailings dams have evolved into extremely large structures. Tailings dams now rank amongst the world's largest dams. The most recent "World Register of Tailings Dams", published by the International Commission on Large Dams (ICOLD), lists 8 tailings dams higher than 150 m, 22 higher than 100 m and 150 higher than 50 m. Six of these impoundments have surface areas greater than 100 km² and storage volumes exceeding $500 \times 10^6$ m³. These dams present essentially the same structural safety problems to those posed by conventional water storage dams with the additional potential concerns from the tailings geochemistry.

Tailings dams are of four basic types. The first three types are shown on Figure 3.2 and begin with a starter dyke constructed of earth, rockfill or even tailings and are then filled with discharge tailings. The last type is essentially a conventional water retention dam. The four dams can be briefly described as:

- **Upstream Constructed Tailings Dams**

As noted earlier, operators in the early days of mining developed a technique for building tailings dams which required a minimum of fill placement. This technique involved construction of an initial low starter dyke and the discharging of tailings from the top of this dyke until the impoundment was filled with tailings. Once full, a second small dyke was constructed with some of the impounded tailings. The procedure is repeated as required and the tailings dam rises in an upstream direction over top of the previously deposited tailings. This method has become known as the *upstream method* of tailings dam construction. *Irrespective of the name provided by the designer/operator, if a vertical line extended downward from the upstream edge of the crest intersects beached tailings the tailings dam in question is an upstream constructed facility. Artificial terminology (e.g. "modified centre-line dams) serve only to confuse the industry and its regulators.*
Figure 3.2 Methods of Tailings Dam Construction
• **Downstream Constructed Tailings Dams**

*Downstream-type* tailings dams are raised by placing additional fill material on the downstream, or outer slope of the previous raise. Dam fill, which can consist of many material types, is not underlain by previously deposited tailings. In addition, the properties of the fill can be specified and controlled to attain specific characteristics. The downstream method achieves a configuration very much like that of a conventional water-retention dam, with similar structural characteristics in most respects.

• **Centreline Constructed Tailings Dams**

The *centreline-type* dam is raised by placing fill on the downstream slope and on the crest of the previous raise. The centreline method shares most of the structural characteristics of downstream-type dams, but uses less fill material to achieve the same height. This is often the most technically and economically viable solution where optimization of these two aspects is desired.

• **Conventional Water Dams**

There are some cases when water may be in direct contact with the inner face of the dam: when floodwaters or seasonal snowmelt must be stored in the impoundment; when a full tailings beach cannot be created; or when a water cover is maintained over the tailings for limiting oxygen diffusivity in an attempt to control acid generation. These types of tailings dams that have this water contact characteristic and that are not raised are known as *conventional water retention-type* dams.

A comparison of the advantages and disadvantages of the 4 basic types of tailings dams is presented in Table 3.2. Figure 3.3 illustrates differences in mechanically-placed fill quantities between the upstream, centreline and downstream dam types. The fill requirement is directly related to fill cost which is largely responsible for the continued use of the traditional but potentially problematic upstream construction method.
Table 3.2  Comparison of Tailings Dam Types

<table>
<thead>
<tr>
<th>TYPE</th>
<th>ADVANTAGES</th>
<th>DISADVANTAGES</th>
<th>GENERAL BEHAVIOUR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream</td>
<td>• smallest dam volume</td>
<td>• failure history</td>
<td>• can be acceptable in static loading</td>
</tr>
<tr>
<td></td>
<td>• revegetation slopes in stages</td>
<td>• requirement for high level of design and construction attention to avoid failure</td>
<td>• generally susceptible to transient loading failures</td>
</tr>
<tr>
<td></td>
<td>• long mining history</td>
<td></td>
<td>• most geotechnically challenging</td>
</tr>
<tr>
<td>Downstream</td>
<td>• most stable</td>
<td>• largest dam volume</td>
<td>• excellent</td>
</tr>
<tr>
<td></td>
<td>• construction control</td>
<td>• revegetate the entire slope at closure</td>
<td>• overtopping and filtration of tailings largest concerns</td>
</tr>
<tr>
<td></td>
<td>• design can be modified to suit performance</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• good safety record</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Centreline</td>
<td>• combines best features of upstream and downstream types</td>
<td>• revegetate most of the slope at closure</td>
<td>• excellent</td>
</tr>
<tr>
<td></td>
<td>• good safety record</td>
<td>• must ensure upstream stability</td>
<td>• overtopping and filtration of tailings largest concerns</td>
</tr>
<tr>
<td>Conventional Water</td>
<td>• good safety record</td>
<td>• often built in one stage</td>
<td>• as per traditional water supply dams</td>
</tr>
<tr>
<td></td>
<td>• conventional design and construction</td>
<td>• high material requirement</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• requires no strength contribution from tailings</td>
<td>• no utilization of &quot;good&quot; tailings in dam structure</td>
<td></td>
</tr>
</tbody>
</table>
Figure 3.3 Comparison of Fill Volumes for Tailings Dam Construction Methods
(adapted from Vick, 1990)
4. PIEZOCONE TECHNOLOGY

4.1 Overview

There are a number of piezocones and their related modules described in this chapter. In addition, other in-situ testing devices that were both research-based and commercially available are also briefly described. To provide appropriate perspective to the descriptions that follow, the contributors of this research to equipment developed are summarized in Table 4.1.

Table 4.1  Development History of Piezocone and Related Equipment Technologies Used in Research

<table>
<thead>
<tr>
<th>IN-SITU TOOL</th>
<th>DEVELOPMENT HISTORY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piezocones (UBC + Hogentogler)</td>
<td>All piezocones used were ASTM standard instruments manufactured prior to research initiation.</td>
</tr>
<tr>
<td>Resistivity Modules</td>
<td>UBC has three resistivity piezocone modules, RES001, RES002, and RES 003. Both RES002 and RES003 were conceived and developed as part of the research. Key advances in RES003 were the isolated circuitry, expanded measurement range and ability to carry out induced polarization measurements.</td>
</tr>
<tr>
<td>Seismic Piezocone</td>
<td>No changes to hardware. Data acquisition significantly upgraded during research tenure.</td>
</tr>
<tr>
<td>Pore Fluid Sampling</td>
<td>KBAT conceived and developed as part of the research.</td>
</tr>
<tr>
<td>Surface Geophysics</td>
<td>Commercial equipment used as available during research tenure.</td>
</tr>
</tbody>
</table>

The two most important equipment advances made during the research were the introduction of the RES003 resistivity piezocone module and the KBAT non-hypodermic water sampling system.

4.2 In-Situ Testing

To provide the best opportunity for a successful subsurface assessment of prevailing physical and geochemical conditions, the selection of the method of site characterization utilized for that assessment is of prime importance. For evaluating mine tailings, traditional methods of limited
drilling and discrete sampling followed by selected laboratory analysis can be expensive and
time-intensive and provide an extremely small statistical sample of subsurface materials.
Furthermore, data quality from the laboratory testing is often suspect due to sample disturbance,
handling errors, mislabeling, etc. In-situ testing, particularly using sophisticated instrumented
and mechanical devices, offers a technical alternative that is arguably more cost effective for
characterization of mine tailings. For the subsurface conditions most common to the mine
tailings, it is the contention of this thesis that in-situ testing using piezocone technology offers a
characterization alternative to materials that are notoriously difficult to sample. Sampling, whilst
maintaining in-situ structure and fabric, followed by attempts at replication of physico-chemical
in-situ conditions in the laboratory has been shown to have limited success on many mine tailings
projects.

The Author carried out his initial research with in-situ testing tools 15 years ago. Over that
period of time, the geotechnical community has come to accept one of the test devices, the cone
penetration test (CPT), as a premier soil-sequence logging tool for most soil conditions in sub-
gravelly materials. Besides providing approximate stratigraphic information the CPT, which
now routinely includes pore pressure measurement (CPTU or, as commonly, piezocone), can also
provide estimates of key geotechnical parameters and information on the physical nature of
groundwater systems. Over the past few years, tools that augment the piezocone have been
developed that allow the hydrogeology and environmental engineering fields to have even greater
benefits from in-situ testing.
4.3 Traditional Tailings Characterization Methods

As defined above, in-situ testing involves evaluating material properties "in-place"; where the effects of physical and chemical stresses on a given soil element are evaluated as is. Conventional drilling and discrete sampling represents a quasi in-situ test method although laboratory evaluations are typically required from retrieved samples to achieve estimates of in-situ engineering parameters. The large experience base and the retrieval of a physical sample are important advantages of the traditional methods whereas lack of standards (or lack of adherence to standards), relatively high cost and questionable sample quality represents the key disadvantages. Impressive strides in numerical modeling over the past ±five years provide good support to laboratory test data. The laboratory data can provide strain-limited backbone information to the numerical model that can then be used to estimate in-situ conditions.

Conventional mud-rotary drilling in mine tailings with reasonable, albeit far from continuous, sampling frequency will yield a 30 metre (100 ft) hole in 8 to 20 hours (and more). The actual drilling production rate will depend upon soil conditions, equipment and operator quality, the method of sampling used and whether there are any cross-contamination of aquifers, decontamination and/or grouting issues or requirements. Although specifying conventional drilling on a metreage basis is never recommended for obvious reasons, from the Author’s experience a generic $150 CDN hourly rate with modest consumables can be used to provide an approximate $40 to $100 per metre for drilling and discrete sampling in mine tailings. These rates are exclusive of mobilization/demobilization costs. The costs of laboratory testing adds further to the effort.
In addition, once a sample is obtained, its quality must be assessed. Table 4.2 presents some of the steps and sources of error in selecting a representative sampling location and then taking a sample from its in-situ condition to the laboratory. Table 4.2 does not include the errors introduced by the laboratory itself, such as sample disturbance, loss of in-situ stresses, and chemical alterations, etc.

Table 4.2  Steps in Sample Procurement and Potential Sources of Error

<table>
<thead>
<tr>
<th>STEP</th>
<th>SOURCES OF ERROR</th>
</tr>
</thead>
<tbody>
<tr>
<td>In-Situ Condition</td>
<td>Soil element exists under specific set of physical and chemical conditions.</td>
</tr>
<tr>
<td>1. Establishing a sample location</td>
<td>Improper borehole completion/placement; poor location choice; poor drilling method choice for soil conditions</td>
</tr>
<tr>
<td>2. Sample collection</td>
<td>Sampling mechanism/procedural bias; operator error</td>
</tr>
<tr>
<td>3. Sample retrieval</td>
<td>Sampling mechanism/procedural bias; loss of confining stress; loss of fabric; sample exposure; degassing; oxygenation; cross-contamination</td>
</tr>
<tr>
<td>4. Preservation/storage</td>
<td>Handling/labeling errors; temperature, etc. control</td>
</tr>
<tr>
<td>5. Transportation</td>
<td>Delay; sample loss; additional disturbance</td>
</tr>
</tbody>
</table>

From the most restrictive viewpoint, therefore, drilling and sampling can be relatively expensive which provides potentially suspect reliability on a non-continuous basis. However, due to its popular use, regulatory acceptance has traditionally been high although recent trends in data quality management and geostatistical evaluations are tending to make it more difficult to convince all regulators that data from drilling, sampling and companion laboratory programs are either comprehensive or accurate enough.

Notwithstanding any of the above, drilling and sampling followed specific laboratory testing will properly remain as essential components of soil (including mine tailings) site characterization.
Just as there are concerns with the available discrete sampling and subsequent testing methods, there is not yet an in-situ testing tool or technique without equal or greater limitations. For the most part, in-situ tools cannot control stress path, alter chemical or hydraulic gradients or alter material state to a predetermined condition to predict future loading conditions. Moreover, fundamental material behaviour cannot be investigated without use of laboratory testing.

The Author’s contention is simple; there is presently no ideal sampling and laboratory testing methodology available for evaluating mine tailings. The strengths and weaknesses of each candidate characterization tool/method need to be clearly understood by individuals in control of selecting the method(s) of site characterization to be utilized. This thesis concentrates on one particular stream of in-situ testing; piezocone technology. The relative merits, and detriments, of piezocone technology which can be compared to more traditional tailings characterization methods are presented in this document. How such information is utilized is highly individual investigator specific and influenced by site conditions and regional standards.

4.4 Standard Piezocone Technology

4.4.1 General

Probing with rods through weaker soils, initially to locate a firmer stratum, has been practised since the early part of this century. It was in the Netherlands in about 1934 that the CPT was introduced in a form recognizable today. The CPT has been referred to as the Static Penetration Test, Quasi-static Penetration Test, Dutch Sounding test and Dutch Deep Sounding Test. The first electronic cone was introduced in 1948 and vastly improved in 1971 (de Ruiter, 1971) when strain gauged load cells were added.
In the modern CPT, a 60° apex and typically 35.7 mm diameter (10 cm² area) cone tip, which resides at the end of a series of rods of the same or lesser diameter as the cone, is pushed into the ground at a constant rate (standard is 2 cm/sec or approximately one metre per minute) and continuous measurements are made of the resistance to penetration of the cone. Measurements are also made of the resistance to penetration of a surface sleeve, which is has a 150 cm² surface area and is located just behind the cone tip. Both dimensions and rate of penetration are controlled by rigorous ASTM and International standards.

In soft soils, cone penetration to depths in excess of 100 metres (330 feet) may be achieved provided verticality (also monitored) is maintained. Gravel layers and boulders, heavily cemented zones and unusually dense sand layers can restrict the penetration severely and deflect and damage cones and rods, especially if overlying soils are very soft and allow rod buckling. This latter scenario can be of concern in thick tailings deposits which directly overly bedrock, glacial till or other similar stratum of substantially stiffer nature than the mine tailings deposit.

One of the most significant developments in CPT technology has been the addition of pore pressure measurements which, as noted previously, is then commonly called a piezocone. The development of piezocone technology has added a new dimension to the interpretation of geotechnical parameters, particularly in loose or soft, saturated deposits. Figure 4.1 shows a schematic cut-away of a typical modern piezocone. Figure 4.1 is per the current University of British Columbia’s (UBC) In-Situ Testing Group (ISTG) “standard” piezocones described in Section 4.4.4. The standard piezocone measures corrected tip resistance (q_t), friction sleeve stress (f_s), and pore pressure response at up to three locations. These pore pressure measurement locations are not explicitly standard but the locations, typically referred to as U1, U2, and U3, are
Figure 4.1 Cut-Away Schematic of Modern Piezocone
most commonly located as shown on Figure 4.1. Temperature \( t \) and inclination \( i \) are also measured simultaneously as the piezocone is advanced into the ground. Inclination is an important measurement in mine tailings where the Author's experience suggests limiting deflections be established at 2° per metre with total deflection of 12° to 20°. Extreme operator experience is required when approaching these limits and tightly-threaded rods are essential. For less-experienced operators, 1° per metre (per rod) or total deflections to 10° are probably better limiting values.

The most common commercially available piezocones, e.g. per those available from Hogentogler Ltd., are “subtraction” type piezocones. A subtraction piezocone has two load cells that measure tip stress and tip stress plus sleeve friction, respectively. The determination of sleeve friction from a subtraction cone is, therefore, not a directly measured value but a difference between two load cells. As the tip stress is typically two-three orders of magnitude larger than the sleeve friction, this would not appear to be an ideal measurement system. However, the subtraction cone has shown good overall robustness and several researchers indicate acceptable accuracy in the recent designs and manufacturing details (e.g. Schaap and Zuijberg, 1982, and Lunne et al., 1997).

Notwithstanding the recent improvements, where accuracy of sleeve friction measurements are of importance, there should be a preference for piezocones with independent measurements of tip resistance and sleeve friction. The most common type of independent measurement piezocone is the “compression” piezocone. In the compression piezocone, the tip and sleeve load cells compress independently under penetration load. The tip load cells are directly in-line with the
conical tip whereas the sleeve load cells communicate to the friction sleeve through a threaded connection.

All of the piezocone channels are continuously monitored and are typically digitally reported at 25 mm to 50 mm intervals, thus providing essentially continuous in-situ data sampling. Regardless of the system used, modern systems allow the data to be acquired in clear format ASCII files. The files allow the user to carry out straightforward post-investigation analyses with any number of proprietary and commercial piezocone evaluation software packages. Additionally, simple programs or spreadsheets can be used to analyze data. Section 4.4.4.2 describes the data acquisition system used for this study.

Davies and Campanella (1995) outline the piezocone’s advantages, limitations, general state of development, as well as typical testing and interpretation procedures. Lunne et al. (1997) offer a comprehensive text on modern industry-level piezocone technology.

4.4.2 Interpretation of Data

Following sections of this thesis address specific interpretative procedures for estimating geoenvironmental engineering properties of mine tailings from piezocone technology. However, for the physical geotechnical parameters, there is a fundamental basis to the interpretative process. This basis is the manner in which the physical piezocone data (e.g. tip, sleeve and pore pressure) are combined to provide estimates of in-situ conditions.
Most piezocone literature makes no attempt to provide a fundamental basis for the parameter estimation techniques offered. This does not necessarily invalidate such literature as the empirical relationships offered are often very useful to practitioners and, under scrutiny, can tend to have some theoretical validity. However, for this research, a consistent method of interpretation was sought which would allow comparing existing and proposed interpretive procedures on a consistent basis. Although beyond the scope of this research, an approach to piezocone interpretation that goes beyond empiricism and/or cavity expansion concepts should be sought (e.g., Wroth, 1988; Been and Jefferies, 1993).

The first step in the interpretive process is to characterize the nature of the piezocone test. There is no question about the complex stress-strain fields created in in-situ soil deposits from the introduction of a piezocone. The piezocone has three independent measurements:

i. Tip resistance;

ii. Sleeve friction; and

iii. Pore pressure response.

In drained penetration, the tip resistance is a measure of modulus based upon concepts of cavity expansion (e.g., Vesic, 1963). Using these concepts, values of frictional strength can be estimated. For undrained penetration, using cavity expansion, the tip resistance and sleeve friction give measurements of undrained strength.
Houlsby (1988) makes an initial argument for the piezocone test to have an analogy to the undrained triaxial compression test. This analogy is explored herein.

In a triaxial test, the initial conditions are the cell pressure $\sigma_3$ and the static pore (back) pressure $u_o$. Initially in the triaxial cell, under isotropic consolidation, $\sigma_3 = \sigma_o$. With the piezocone, the comparative initial conditions are the overburden stress, $\sigma_{vo}$ and the static pore pressure $u_o$. To be most appropriately rigorous, some (e.g., Been and Jefferies, 1985; Wroth, 1988) argue for use of the in-situ horizontal stress or the mean stress versus the overburden stress. However, the piezocone cannot provide reliable indications of lateral stress conditions so it is convenient to use the vertical (overburden) stress. In-situ static pore pressures can be readily obtained from the piezocone under most circumstances.

In the undrained triaxial test, a stress increment ($\Delta \sigma_1$) is applied to the cell and these stress increments and the corresponding change in sample pore pressure are measured. Quantities commonly obtained from the test are the maximum stress at failure ($\sigma_1 - \sigma_3$) and the difference in pore pressures from the initial condition ($u_f - u_o$). Folding the stresses and pore pressures together to provide effective stress measurements is the most common convention, with effective stress difference at failure and the initial effective stress conditions often being used to describe soil behaviour. Two useful combinations of parameters from the triaxial test are:

$$\frac{\sigma_1 - \sigma_3}{\sigma_3 - u_o}$$

and

$$44$$
The ratio in equation 4.1 can be related to Mohr-Coulomb effective stress strength values from an equivalence to $2 \cdot \sin \phi (1 - \sin \phi)^{-1}$. Equation 4.2 is Skempton's pore pressure parameter at failure, $A_f$, and can be related to sample consolidation history.

For the piezocone, there are three independent measurements of in-situ condition; tip ($q_t$), sleeve ($f_s$) and pore pressure ($u$). The tip and pore pressure, $q_t$ and $u$, are respectively analogous to $\Delta \sigma$, and $u_f$ in the triaxial test. To push the analogy further, equations 4.3 to 4.5, inclusive, present the three key stress differences with the piezocone:

\[
\frac{(u_f - u_o)}{(\sigma_1 - \sigma_3)}
\]  

(4.2)

\[
(q_t - \sigma_{vo})
\]  

(4.3)

\[
(u - u_o)
\]  

(4.4)

\[
(\sigma_{vo} - u_o)
\]  

(4.5)

The sleeve friction, $f_s$, can be used alone as an indication of in-situ shear behaviour but $q_t$ and $u$ cannot be used alone in describing in-situ stress conditions.

The fundamental basis noted above is to take the key stress differences from the piezocone and develop dimensionless ratios. Dimensionless, or stress normalized, parameters allow unified soil classification and parameter estimation as will be described later in the thesis. This is analogous
to the normalized parameters/ ratios developed in the laboratory from triaxial testing. This stress normalization is as per the triaxial test with the piezocone analogous ratios being:

\[
Q = \frac{q_t - \sigma_{vo}}{\sigma_{vo} - u_o} \quad (4.6)
\]

\[
B_q = \frac{u - u_o}{q_t - \sigma_{vo}} \quad (4.7)
\]

In addition, friction ratio values (ratio of the sleeve friction to the tip resistance) should be stress normalized as:

\[
F = \frac{f_s}{(q_t - \sigma_{vo})} \quad (4.8)
\]

It is important to note that non-normalized friction ratio values are almost always numerically equivalent (to significant digits) to normalized values due to the usual large stress difference in equation 4.3 except under extreme conditions. As per non-normalized values, \( F \) from equation 4.8 is most commonly quoted as a percentage for convenience.

Further analogy to the triaxial test can come from combining equations 4.6 and 4.7 into equation 4.9 as follows:

\[
\frac{(q_t - u)}{(\sigma_{vo} - u_o)} = Q (1 - B_q) - 1 \quad (4.9)
\]
Equation 4.9 is analogous to the principal effective stress ratio, $\sigma'_1/\sigma'_3$. Equation 4.9 is key to the suggested soil classification procedure described in Section 6.

### 4.4.3 Typical Engineering Uses

Equivalent stratigraphic logging with the piezocone is one of its primary uses in site investigation work. As the cone is advanced, the forces measured by the tip and the friction sleeve vary with material properties of the soil being penetrated. The excess pore pressure ($\Delta u$) measured during penetration is also useful indication of soil type and provides another excellent means of detecting details in soil layering. The best interpretation methods combine both the tip and sleeve interpretation with some type of pore pressure interpretation. Section 6 will present a proposed stress normalized interpretation chart for tailings (and soils in general) based on dimensionless cone bearing, friction ratio, both of which are normalized with respect to overburden stress, and dynamic pore pressure. As will be shown later, the need to stress normalize piezocone measurements is always important but becomes essential to adequate soil behaviour characterization where overburden stress exceed about 200 kPa. As many tailings facilities include stress regimes with overburden pressures in excess of 200 kPa, normalization is considered essential for characterization work with mine tailings. Combined interpretation using tip, sleeve and pore pressure measurements allows very comprehensive soil behaviour logging with layer discernability in the order of a few centimeters.

Excess pore pressure measurements can provide valuable insight into the hydraulic parameters of the porous media. When penetration ceases, e.g., after a 1-metre rod push, any excess pore pressures generated during cone penetration will start to dissipate. The rate of dissipation is
dependent upon the coefficient of consolidation, which, in turn, is dependent upon the compressibility and hydraulic conductivity of the soil. Estimates of the vertical and horizontal coefficients of consolidation, \( c_v \) and \( c_h \), respectively, for silty to clayey soils may be obtained by monitoring the rate of dissipation of the excess pore pressure. In addition, pressure head distribution within the saturated zone can be estimated based on the equilibrium pore pressure data for all soil types. Estimates of hydraulic conductivity from piezocone pore pressure decay in sandy material, however, are more difficult because dissipation occurs very rapidly.

Borrowing from experience in soil site testing, it is becoming accepted that the piezocone can be used to estimate the geotechnical and hydrogeological parameters, shown in Table 4.3 with a fair degree of accuracy.

**Table 4.3 Common Parameters Estimated with Piezocone Data**

<table>
<thead>
<tr>
<th>Drained Penetration (e.g., sands and some silts)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>• relative density, ( D_r )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• friction angle, ( \phi )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• deformation moduli such as ( M, E ) and ( \sigma_{\max} )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Undrained Penetration (e.g., clays and most silts)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>• undrained shear strength, ( s_u )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• sensitivity, ( S_t )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• stress history, OCR</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• deformation moduli such as ( M, E_u, \sigma_{\max} )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Dissipation of Dynamic Pore Pressures</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>• coefficient of consolidation, ( c_h )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• hydraulic conductivity, ( k_h )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• equilibrium water pressure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• hydraulic gradients</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Other common uses for piezocone data include:

- assessing liquefaction susceptibility;
- estimating equivalent SPT N value;
- estimating friction pile capacity; and
- ground improvement quality control.

Parameter estimates have traditionally been obtained on either a purely empirical basis, through theoretical considerations and/or correlations with laboratory test results, large chamber test results, and/or relationships with other relevant in-situ tests.

4.4.4 Piezocones Used in This Study

4.4.4.1 Piezocone Types

The UBC-ISTG utilizes both its own-cones and commercially available piezocones from Hogentogler Co. of Maryland, USA. The UBC-ISTG's cones are designed, machined and instrumented at UBC. This development process provides the UBC-ISTG with a unique perspective on the limitations and potential of the modern piezocone. During the tenure of the research, the cones summarized in Table 4.4 were used by the Author at the sites described in Section 5.

Table 4.4 Piezocones Utilized in This Research

<table>
<thead>
<tr>
<th>CONE NAME</th>
<th>MANUFACTURE</th>
<th>TYPE</th>
<th>SPECIAL FEATURES</th>
</tr>
</thead>
<tbody>
<tr>
<td>HOG 2</td>
<td>Hogentogler</td>
<td>Subtraction: $q_c$ and $q_c+f_s$</td>
<td>Standard commercial piezocone</td>
</tr>
<tr>
<td>UBC 9</td>
<td>ISTG</td>
<td>Independent compression: $q_c$ and $f_s$</td>
<td>U1, U2, U3 capability</td>
</tr>
<tr>
<td>UBC 10</td>
<td>ISTG</td>
<td>Independent Compression: $q_c$ and $f_s$</td>
<td>U1, U2, U3 capability</td>
</tr>
</tbody>
</table>
Further details on cone type can be found in Lunne et al. (1997). It is, however, important to note that the subtraction-type piezocones of the mid to late 1990s are comparable in accuracy to compression-type cones; a situation not available in the 1980s.

4.4.4.2 Data Acquisition

During the tenure of the research, the UBC-ISTG abandoned a commercially available data acquisition system (DAS) in favour of our own system. GEODAS was conceived and developed by the UBC-ISTG to provide a more comprehensive DAS for applied research. The program is fully interactive and provides a real-time graphical user interface (GUI). The GUI is extremely useful for more specialized tests such as seismic, resistivity and induced polarization piezocone testing described later in this section.

4.4.4.3 Calibration

Prior to commencing any detailed field program with the piezocone, a calibrative check of the various cone channels was carried out. The UBC-ISTG cones are originally calibrated over their full range once appropriately exercised. The calibration values for UBC 9 and UBC 10 were established, therefore, in the UBC-ISTG electronic laboratory. The cone tip and sleeve are check-calibrated by applying several known loads to the cone and measuring corresponding voltage outputs. The voltage outputs are then compared to the expected calibrated values under the given loads. As an example, calibration accuracy is acceptable when within 0.07% of full scale for the cone tip (0.7 bar for the 1000 bar cones used in this research).
During field programs, when abrupt conditions of load and/or more than 50% of the full-scale load was applied, the cones were again check-calibrated. The cones were also checked for tip, sleeve and pore pressure calibrations every five field days regardless of loading conditions. The manufacturer initially calibrates the pore pressure transducers. Check-calibrations for these transducers are carried out by placing the cone into a sealed calibration chamber and then increasing the air pressure to known values. The other channels were calibrated following specific procedures on an as-needed basis.

4.5 Seismic Piezocone

Starting in the mid-1980's, equipping piezocones with seismic pick-ups resulted in a tool, the seismic piezocone, that can be used for a procedure known as the seismic cone penetration test (SCPTU). Small strain wave velocities, and more recently in-situ damping ratio, can be determined in an accurate, rapid and highly repeatable fashion with the SCPTU. Beyond the advantage of retaining all of the information available with the standard piezocone, a further attraction of seismic cone technology is the much lower cost involved than standard downhole or crosshole geophysical seismic methods. Campanella and Robertson (1984) first reported the addition of a seismic pick-up in a standard piezocone. The seismic piezocone allows downhole seismic techniques to be carried out with a surface shear source during a pause in cone penetration; typically at the 1 metre rod breaks. The seismic pick-up in a seismic piezocone, either a geophone, which measures particle velocity, or an accelerometer (depending upon the application), is located within the standard piezocone as shown schematically in Figure 4.1.
Virtually all soils show non-linear stress strain behaviour even at very low strains. Consequently, adequate knowledge of the governing constitutive behaviour is essential for addressing many non-brittle geotechnical-engineering problems. A typical non-linear stress strain relationship for many soils can be approximated by hyperbolic curves; a constitutive relation that can be established with knowledge of the maximum shear modulus and the shear strength. As most soils are strain softening at large enough strains, shear modulus typically decreases with increasing shear strain. However, the shear modulus is almost always constant at shear strains less than $10^{-4}\%$ and is generally referred to as the dynamic shear modulus, $G_{\text{max}}$ at these low strains. Shear modulus is a fundamental soil property which relates shear deformation to shear loading. With the shear strength being determined from the piezocone as noted in the previous section, a rudimentary stress-strain curve can be developed for most soils with the seismic piezocone.

Using elastic theory one relates the maximum shear modulus, $G_{\text{max}}$, shear velocity, $V_s$, and total mass density, $\rho$, as:

$$G_{\text{max}} = \rho V_s^2$$  \hspace{1cm} (4.10)

The typical arrangement utilized in this research for conducting the seismic piezocone test is shown schematically in Figure 4.2. For routine shear (or compression) wave velocity measurements, neither calibrated input energy nor detailed signal processing is required and the velocities can be easily determined from arrival times. However, for damping measurements, it is necessary to evaluate the quality and nature of the signal that the shear wave produces in
Figure 4.2  Typical Arrangement for Conducting a SCPTU

\[ V_s = \frac{L_2 - L_1}{t_2 - t_1} \]
advance of velocity and other calculations (Stewart and Campanella, 1993). Signal processing also leads to a better understanding of the properties of the waves.

4.6 Resistivity Piezocone

4.6.1 General Concept

The measurement of the electrical properties of earth media was first carried out between 1900 and 1905 by Conrad Schlumberger. These initial efforts involved injecting electrical currents into the ground and measuring the resultant potential field distribution. Since that time, electrical investigations for subsurface purposes has come to represent a very key field in applied geophysics.

The electrical resistivity/conductivity of earth media is a measurement of the difficulty/ease with which an electrical current can be carried through that media. This simple concept can be illustrated using Figure 4.3 where \( R \) = resistance and \( G \) = conductance. In Figure 4.3, a tank is present that, when empty (air filled), no conductance of electrical current is possible and the ammeter will read zero current regardless of the amount of voltage supplied. The tank can then be filled with a variety of earth materials alone, or in mixture with, a variety of pore fluids. In each case, a different, and finite, current will be measured for a given voltage potential supplied. If a number of different sized tanks were used, the measured current (i.e. the measured resistance to current flow) would be noted to be a function of tank length and of the area of the conductive end plates. There is also a constant of proportionality, which is independent of tank dimension, which is defined as the system’s resistivity. In MKS, resistivity is given the units of ohm-m.
CONDUCTIVE END PLATES (AREA A)

AMMETER (CURRENT I)

BATTERY (VOLTAGE V)

Figure 4.3  Concept of Resistivity/Conductivity (adapted from McNeill, 1980)

**Resistivity** ($\rho$)

$$\rho = \frac{RA}{L} \text{ OHM-METRES}$$

WHERE $R = \frac{V}{I} \text{ OHMS}$

**Conductivity** ($\sigma$)

$$\sigma = \frac{GL}{A} \text{ MHOS/METER}$$

WHERE $G = \frac{I}{V} \text{ MHOS}$
The reciprocal of resistivity is *conductivity*. Just as resistivity, \( \rho \), is related to resistance, \( R \), conductivity, \( \sigma \), is related to conductance, \( G \), by a constant of proportionality. Fluid conductivity meters and most commercial electromagnetic instruments typically measure conductivity, not resistivity. However, they are always inversely related and can be equally well stated as either as long as the units are consistent.

Most soil particles represent very effective electrical insulators (i.e. very high resistance to current flow). However, in metallic mine tailings (e.g. pyritic ore bodies) this general rule can be an overly simplistic assumption. In general, the nature of the passage of electrical current through the soil-fluid system is electrolytic with the insulating soil particles playing little role in the conductance of current other than to increase resistance. The resistivity/conductivity measured for soil systems is governed by:

- the porosity of the soil matrix;
- the shape and size of the pore spaces including number and size of interconnecting passages;
- the degree of saturation;
- concentration of dissolved electrolytes in pore fluid;
- temperature and phase-state of the pore fluid; and
- amount and composition of colloids.

Archie (1942) presents an empirical relationship to capture the essence of the pore fluid-bulk material resistivity interaction. This relationship is discussed in Chapter 10.
For soil conditions such as mine tailings, the major influences on measured resistivity/conductivity are the degree of saturation and the nature of the pore fluid electrolytes. The conductivity of an electrolyte is proportional to both the total number of ions present and their respective velocities (mobilities). The most mobile ions are H\(^+\), OH\(^-\), SO\(_4\)\(^-\), Cl\(^-\), K\(^+\) and NO\(_3\)\(^-\); however, of these, the hydrogen and hydroxyl ions are easily the most mobile.

4.6.2 Development of the Research Tool Used

The resistivity piezocone (RCPTU) is a relatively recent development in piezocone technology (Campanella and Weemees, 1990). RCPTU commercial use is limited, but increasing (e.g., Horsnell, 1988; Woeller et al., 1992; and Rossabi, 1993). The ability to measure the resistance to current flow in the ground on a continuous basis is extremely valuable due to the large effects that dissolved and free product constituents have on soil resistivity (conductivity). The RCPTU consists of a resistivity module which is added behind a standard piezocone. Davies and Campanella (1995) give an overview summary of the RCPTU and its perceived application areas.

As noted in the background information, the entire concept of the RCPTU is that the measurements of bulk resistivity trends indicate whether some form(s) of dissolved or free product constituents are at or above background values. Background values are established either from on-site and/or regional experience or from similar geological environments. The areas where background values are exceeded are then further evaluated with appropriate groundwater sampling at discrete depths for in-depth chemical analysis. The combination of RCPTU screening with discrete water sampling provides a rapid, cost-effective means of carrying out geoenvironmental site characterizations in many soil conditions (Davies and Campanella, 1995).
The bulk resistivity is not directly measured by the resistivity module, but rather it is determined from the measured alternating current (AC) voltage \( V \) across an electrode pair at a constant supplied current \( I \) at 1000 Hz. At 1000 Hz, there do not appear to be any frequency effects on the measured resistivity values where such values are less than 1000 Ω-m and the direct current (DC) analogy of Figure 4.3 will apply. The bulk resistance of the soil \( R \) is computed from Ohm's Law:

\[
R = \frac{V}{I}
\]  (4.11)

The bulk resistance is not a fundamental property of the soil-pore water system. It is dependent upon the current path length \( L \) and the cross-sectional area \( A \) of the effective resistive unit. The bulk resistivity can be computed from the bulk resistance if the following assumptions are made:

- the soil acts as a homogeneous isotropic media;
- the measurement electrodes act as perfect conductors; and
- the resistivity module circuitry acts as a perfect current supply source.

Unlike bulk resistance, bulk resistivity \( \rho \) is a fundamental property of the porous media and is related to the bulk resistance in the following manner:

\[
\rho = \frac{A}{L} \times R = CF \times R = CF \times \frac{V}{I}
\]  (4.12)
The calibration factor \((CF)\) of the resistivity module is dependent upon the geometry of the electrode dimensions and the magnitude of the excitation current. \(CF\) is a constant for a given configuration of electrode spacing and excitation current, and is determined by submerging the resistivity module in a constant temperature buffer solution of known resistivities (conductivities).

When the electrodes are in a homogeneous and isotropic medium, their electrical response will be in a similar manner to that observed in the calibration procedure. However, soil is rarely homogeneous and isotropic, so during field-testing the response of the electrodes will be dependent on the state of the soil and the changes to this state caused by the penetration process. However, of considerable practical value is the fact that the measured resistivity is almost totally governed by the pore fluid chemistry and the pore volume. In other words, soil chemistry has a limited effect in most circumstances. Just as all resistivity/conductivity tools, the value of bulk resistivity measured by the RCPTU (or any other non-intrusive bulk measurement electrical tool) is mainly effected by:

- the degree of fluid saturation;
- water chemistry (fluid chemistry); and
- soil chemistry (solid conduction).

For most cohesionless soils, the water chemistry dominates the resistivity value. The only exceptions here are where the surface charges, either negative or positive, on the individual mineral grains can become quite high. This only occurs in metallic materials such as pyrite and
other highly conductive metals. In most geoenvironmental problems, the assumption of water chemistry dominance in the bulk resistivity value is valid. However, as noted above, in mine tailings this may not be appropriate if a metallic mine is involved although the effect may be relatively constant.

For cohesive soils, where clayey particles become a large percentage of the overall soil matrix, soil chemistry can have a large effect on the measured bulk resistivity. The surface charge on clays is fixed (i.e., unlike mineral soils, independent of pH) by lattice substitution and is essentially always negative. This leads to an electrical double layer which, in clayey soils, affects the electrical, mechanical and chemical properties of the soil. In general, the higher the surface area to volume ratio of a soil the greater the surface conductivity effect will be. In clays this surface effect can become dominant.

During a RCPTU sounding, the electrodes will not respond fully to a layer unless the layers are completely within the electrode spacing. For minimum layer thickness to be correctly sensed the thickness must be greater than the electrode spacing. Smaller distances between the electrodes allow for the possible detection of thinner layers of contrasting resistivity. Wider spacing provides an average resistivity over a larger depth and a greater penetration of the electric field into undisturbed soil. This should give a more accurate determination of soil resistivity in homogeneous ground.
A schematic of the original UBC-ISTG resistivity piezocone module is shown in Figure 4.4. Figure 4.4 shows RES001, a module described in the following section. An excitation current of typically 1000 Hz is supplied to the outer electrodes, and the bulk resistivity of the soil is measured across up to three pairs of electrodes. The smallest electrode spacing is useful for detection of thin layers of contrasting bulk resistivity, whereas the largest electrode spacing measures an average resistivity over a larger depth and allows a greater lateral penetration of the electric field into the undisturbed soil.

Keys (1989) notes that the depth of penetration from this type of logging device is roughly twice the electrode spacing. Tank calibration tests with the resistivity modules described in the following sections generally agreed with this observation. The resistivity piezocone shown schematically in Figure 4.4 therefore has penetration capability of about 20 and 150 mm for the inner and outer electrodes respectively. A potential enhancement to this depth of penetration with RCPTU technology is the use of focused resistivity concepts and further research in this area is suggested.

One interesting aspect of the resistivity module for piezocone testing is related to its size. The diameter of the module must be greater than the piezocone itself to ensure intimate soil-electrode contact. However, ASTM D-3441 notes that push rods should not have an outside diameter greater than that of the base of the piezocone for a length of 0.4 m behind the tip or 0.3 m above the top of the friction sleeve. For all of the UBC-ISTG resistivity modules, this ASTM requirement is met.
Figure 4.4 Schematic of RES001
4.6.3 Evolution of the RCPTU During this Research

For this research, only resistivity modules conceived and manufactured by the UBC-ISTG were utilized. In 1992 when the research for this thesis was initiated, the UBC-ISTG resistivity module was RES001, a slight modification of the instrument used by Weemees (1990) in his M.A.Sc. work; i.e., the “first” generation UBC-ISTG resistivity module. Campanella and Weemees (1990) describe this tool and the initial UBC-ISTG work with it. Figure 4.4 shows a schematic of RES001 in the form used in the mine tailings research.

RES001 is a non-isolated resistivity module. This means is that the charging and measurement electrodes can interact with the metallic housing of the module creating a complex electromagnetic field. The analogy is not unlike a partial short-circuit in a linear array. Although consistent measurements are possible with this configuration, the effective range of the instrument is hampered and tool calibration (and subsequently initializing the DAS) is more difficult as discussed in Section 4.6.4.

In 1993, RES002 was conceived and developed. As shown on Figure 4.5, the RES002 module is 15 cm in diameter as was RES001. Initially developed with non-isolated circuitry, this tool was modified in 1994 to have isolated circuitry. The length and diameter combination of RES002 was found to limit effective penetration in denser tailings; i.e., the module was simply too large for a 10 tonne pushing capacity.

The culmination of the research with respect to resistivity modules was RES003 shown schematically in Figure 4.6. A simpler module than RES002 (e.g., only two sets of electrodes), it
Figure 4.5  Schematic of RES002
Figure 4.6    Schematic of RES003
features isolated circuitry and a shorter housing which is only 10.5 cm in diameter. Penetration with the RES003 module was much easier than with RES002 and the smaller diameter design is considered a vast improvement.

4.6.4 Calibration

Calibration of the resistivity modules was generally carried out in a specially designed tank as shown schematically on Figure 4.7. Distilled (up to 24 hours) tap water was initially introduced into the tank and left to reach an equilibrium temperature. Equilibrium was considered to be achieved when the temperature was ±0.1°C over any given 30 minute interval.

As distilled water is extremely insulating, it typically represented the upper end of the resistivity measurements. Increases in tank conductivity were achieved by the introduction of either KCl or NaCl. After each increment of salt loading, the solution was fully mixed and the temperature monitored throughout. Figure 4.8 shows the generally accepted impact temperature has on the measurement of fluid conductivity. Several trials of considerably warmer and cooler than room temperature water confirmed the trend in Figure 4.8. Measurements of fluid conductivity were taken with a portable conductivity metre, which was itself calibrated on a regular basis.

Calibration of the modules results in a series of data in the form of measured potential difference from a given pore fluid resistivity (1/conductivity). As noted, the tank temperature was also monitored throughout testing and, if necessary, adjusted to keep the temperature for calibration to ±0.1°C.
Figure 4.7  Resistivity Module Calibration Tank
Figure 4.8   Effect of Temperature on Pore Fluid Conductivity
Figure 4.9 shows typical calibration from the first generation module, RES001. As shown in Figure 4.9, two levels of voltage excitation (controlled externally by an AC sinusoidal signal generator) were used both in calibration and in actual field-testing. The level of excitation for RES001 is manipulated “real time” in the field by the operator in response to range of resistivity values being encountered during the sounding.

There are two important trends to notice with Figure 4.9:

1. there is only a limited range of resistivity values over which linearity of response can be assumed to approximate the potential difference-resistivity relationship; and

2. that approximate linearity of response does exist between supplied potential difference and fluid resistivity and occurs at low values of resistivity (high values of conductivity or at higher levels of induction).

As shown later in this thesis, for most conductive fluid environments (e.g. most mine tailings and other higher TDS environments), this linearity at the lower end of the resistivity scale is advantageous. However, as noted in other UBC-ISTG research carried out, and described in Everard (1995), insulating contaminants in groundwater systems (e.g. carbon-based non-aqueous and aqueous phased entities) can produce bulk resistivity values in the order of 400 Ω-m to 1000 Ω-m, and greater. Consequently, RES001 was not assumed to be a fully effective dual range instrument. In addition, and not readily apparent in viewing Figure 4.9, is that RES001 lost linearity at the higher excitation voltage in very conductive pore fluid situations. Bulk pore fluid resistivities in high TDS environments can often be less than 0.1 Ω-m. RES001 was generally not able to provide effective linearity below 2 to 3 Ω-m.
Figure 4.9 Typical Calibration Data from RES001
The development of RES002, aimed at extending the effective measurement ranges both higher and lower than RES001, was a significant advance for the UBC-ISTG. Figure 4.10 presents typical calibration data from RES002. Excitation voltage was available over the range of ±3.75 V and the effective range of measurement effectively pushed to about roughly 0.7 Ω-m to more than 400 Ω-m. The upper range was potentially useful to nearly 800 Ω-m although linearity of relationship was lost. The 0.7 Ω-m lower range corresponded to a conductivity of roughly 14 200 µS/cm which, for bulk systems, is nearing the lower range of saturated environments with high total dissolved solids, TDS (e.g. sulphide ore-body tailings with high levels of dissolved metals from oxidation processes). However, as noted previously, the diameter and length combination of RES002 was such that penetration of denser materials was difficult. Consequently, the significant advances over RES001 (e.g. greater measurement range, isolated circuitry, more linearity for calibration factors etc.) were effectively unavailable in many practical situations. It was generally found that any layer with Q>150 (see Chapter 5) and over 2.0 m in thickness was enough to stop penetration with 10 tonnes of pushing force.

The culmination of the research, from a resistivity module development perspective, came with the development of RES003. It was determined that the use of three excitation voltages with RES003 would allow measurement coverage over a very practical measurement range. Figure 4.11 shows typical low excitation voltage (0.1 V) calibration data. Note on Figure 4.11 that the effective upper range is well over 1000 Ω-m. The practical limitation of the insulating properties of the distilled water used limited the typical calibration range. If higher values were of interest (for, example, organic contamination investigation work), miscible insulating
LOW EXCITATION VOLTAGE (0.50 V)
TEMPERATURE = 10 DEGREES C

Figure 4.10 Typical Calibration Data from RES002
Figure 4.11  Typical Low Excitation Calibration Data from RES003
fluids would need to be used for calibration purposes. Figure 4.12 shows medium range excitation range calibration data for RES003. For the tailings research, this excitation range was found very effective for the unsaturated upper zone of the tailings where accurate bulk measurements were desired but where higher excitation voltages were not useable. Figure 4.13, on the other hand, shows the most typical range for use in higher TDS mine tailings. At an excitation voltage of about 5.0 V, very accurate measurements down to a fraction of 1 Ω-m was found to be possible. For example, as shown on Figure 4.14, the two measurement electrodes show good linearity to values of less than 0.2 Ω-m which covered all but the most conductive of environments experienced during the research.

By the end of the research, the combination of RES003 and the improved UBC-ISTG GEODAS allowed three excitation ranges (slopes and intercepts) to be used for calibration factors. Check calibrative efforts typically involved 10 to 30 data points with emphasis on the higher conductivity range where the input voltages were at their upper range. As the tailings research was mainly involved with metallic mine tailings including sulphide tailings, there was a real need to measure much lower resistivities than any previous RCPTU experience.

In the preceding discussion, linearity of response has been emphasized as the benchmark for acceptable calibration over a given excitation voltage range. This linearity is stressed for two main reasons:
Figure 4.13  Typical High Excitation Calibration from RES003
Figure 4.14 Expanded Scale High Excitation Calibration from RES003

Example Calibration Fit

R015 Y = 0.1075X + 3.688
R150 Y = 0.1399X + 3.688

Potential Difference (V)

Resistivity (Ω-m)

-3.40
-3.50
-3.60
-3.70
-3.80
-3.90
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1. calibration factors can then be established over larger ranges of electrical response; and

2. manipulations in GEODAS, both in algorithm and in actual analog to digital conversion, can be appropriately simplified.

The Author is aware of a commercial resistivity module that uses a piece-wise linear calibration with more than 40 linear “pieces”. This commercial module also has automatic excitation voltage switching to meet the appropriate range of measurement being encountered. However, the UBC-ISTG obtains much more data (more frequent sampling rate and more channels) than commercial applications limiting the acquisition speed of GEODAS. As A/D hardware components reduce in price per relative speed of acquisition, the research modules could indeed be calibrated with either smooth function formulae or complex piece-wise linear calibration curves.

For the UBC-ISTG modules, Figure 4.9 showed a typical calibration result for RES001 (non-isolated) whereas Figure 4.11 showed a typical result for RES003 (isolated). Since the calibration factor, CF, is established from the slope of the line, the improvement in calibration (and hence data acquisition) simplicity with the isolated circuitry is clear in comparing these two figures.

4.6.5 Typical Values

As will be shown in Section 11, there was a wide range of resistivity values found in the mine tailings investigated. However, in all cases the trends were consistent and the results numerically comparable with sampled pore water conductivities when an appropriate mixing law was applied.
Figure 4.15 shows a generic trend response of bulk resistivity-conductivity for investigations carried out in an otherwise freshwater sand deposit affected by conductive or insulating phenomenon due to either induced geochemical changes (e.g., higher ionic loading) or changes in geological character (e.g., presence of clay or change in saturation state). The key trends to note from this schematic example are as follows:

- above the water table (or where saturation is less than 98% to 100%), the resistivity is almost always in excess of 150 Ω-m regardless of pore fluid chemistry or mineralogical effects;
- “freshwater” sand bulk resistivity values are generally in the range 70 to 120 Ω-m;
- even modest total dissolved solids (TDS) values (e.g., ≥5000 mg/l) will result in a very detectable bulk resistivity-conductivity response; and
- increases in bulk resistivity at depth above “typical” background values is either due to loss of saturation (e.g., first saturated response due to a perched aquifer) or the presence of an insulating contaminant such as a hydrocarbon.

Table 4.5 presents typical values of bulk soil resistivity measurements with the RCPTU and their corresponding measurements of pore fluid resistivity. The results in Table 4.5 were compiled by the UBC-ISTG over the period 1988 to 1997. Since conductivity is the reciprocal of resistivity it is easy to convert according to:

\[
\text{Conductivity (µS/cm)} = 10,000 / [\text{Resistivity(Ω-m)}] \tag{4.13}
\]
Figure 4.15  Schematic Bulk Resistivity-Conductivity Response Freshwater Sand Deposit
Table 4.5 gives corresponding values in units of conductivity due to their popular use in the geochemical field. These results show that the range of resistivity (conductivity) values is very large from about 0.01 (1000000) to about 1000 Ohm-m (10 μS/cm) and is very sensitive to both soluble salts and low solubility organic contaminants.

Table 4.5  Summary of Typical Resistivity (Conductivity) Measurements of Bulk Soil Mixtures and Pore Fluid (saturated mixtures only) (adapted from Davies and Campanella, 1995)

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Bulk Resistivity $\rho_b, \Omega$-m</th>
<th>Fluid Resistivity $\rho_f, \Omega$-m</th>
<th>Bulk Conductivity $\mu$S/cm</th>
<th>Fluid Conductivity $\mu$S/cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sea water</td>
<td>---</td>
<td>0.2</td>
<td>---</td>
<td>500000</td>
</tr>
<tr>
<td>Drinking water</td>
<td>---</td>
<td>&gt;15</td>
<td>---</td>
<td>&lt;665</td>
</tr>
<tr>
<td>Distilled water</td>
<td>150</td>
<td>60</td>
<td>67</td>
<td>167</td>
</tr>
<tr>
<td>McDonald Farm site (Richmond) clay</td>
<td>1.5</td>
<td>0.3</td>
<td>6700</td>
<td>33300</td>
</tr>
<tr>
<td>Laing Bridge site (Richmond) clay</td>
<td>20</td>
<td>7</td>
<td>500</td>
<td>1430</td>
</tr>
<tr>
<td>Colebrook site (Langley) clay</td>
<td>25</td>
<td>18.2</td>
<td>400</td>
<td>550</td>
</tr>
<tr>
<td>TC @ 232 Ave (Langley) clay</td>
<td>8</td>
<td>---</td>
<td>1250</td>
<td>---</td>
</tr>
<tr>
<td>Strong Pit (Abbotsford) clay</td>
<td>35</td>
<td>---</td>
<td>285</td>
<td>---</td>
</tr>
<tr>
<td>Kidd 2 site (Richmond) clay</td>
<td>14</td>
<td>12.5</td>
<td>715</td>
<td>800</td>
</tr>
<tr>
<td>McDonald Farm site (Richmond) sand</td>
<td>5-20</td>
<td>1.5-6</td>
<td>2000-500</td>
<td>6700-1670</td>
</tr>
<tr>
<td>Laing Bridge site (Richmond) sand</td>
<td>5-40</td>
<td>1.5-10</td>
<td>2000-250</td>
<td>6700-1000</td>
</tr>
<tr>
<td>Colebrook site (Langley) sand</td>
<td>70</td>
<td>---</td>
<td>143</td>
<td>---</td>
</tr>
<tr>
<td>Strong Pit site sand</td>
<td>115</td>
<td>---</td>
<td>89</td>
<td>---</td>
</tr>
<tr>
<td>Kidd 2 site (Richmond) sand</td>
<td>1.5-40</td>
<td>0.5-21</td>
<td>6700-225</td>
<td>20000-475</td>
</tr>
<tr>
<td>Typical landfill leachate</td>
<td>1-30</td>
<td>.5-10</td>
<td>10000-330</td>
<td>20000-1000</td>
</tr>
<tr>
<td>Mine tailings site (base metal) with oxidized sulphide leachate</td>
<td>0.01-20</td>
<td>.005-15</td>
<td>1000000-500</td>
<td>2000000-670</td>
</tr>
<tr>
<td>Mine tailings site (base metal) without oxidized sulphide leachate</td>
<td>20-100</td>
<td>15-50</td>
<td>145-100</td>
<td>665-200</td>
</tr>
<tr>
<td>Arsenic contaminated sand and gravel</td>
<td>1-10</td>
<td>.5-4</td>
<td>10000-1000</td>
<td>20000-2500</td>
</tr>
<tr>
<td>Industry site-inorganic contaminants in sand</td>
<td>0.5-1.5</td>
<td>0.3-0.5</td>
<td>20000-6500</td>
<td>33000-20000</td>
</tr>
<tr>
<td>Industrial site - creosote contaminated silts and sands</td>
<td>200-1000</td>
<td>75-450</td>
<td>50-10</td>
<td>135-22</td>
</tr>
<tr>
<td>Industrial site - organic contaminants in sand</td>
<td>125</td>
<td>---</td>
<td>80</td>
<td>---</td>
</tr>
<tr>
<td>BC Place Parcel 2, PAHs (coal gas plant)</td>
<td>200-300</td>
<td>---</td>
<td>50-33</td>
<td>---</td>
</tr>
<tr>
<td>BC Place Parcel 2 (wood waste)</td>
<td>300-600</td>
<td>---</td>
<td>33-17</td>
<td>---</td>
</tr>
</tbody>
</table>
4.6.6 Additional Electromagnetic Information

Besides analogue resistivity/conductivity measurements, the resistivity modules developed during this research have other potential electromagnetic measurement capabilities. In coming years, it is expected these capabilities will come to be exploited.

The potential for Induced Polarization (IP) measurements with the resistivity piezocone is a recent addition and a unique UBC-ISTG development to the Author's knowledge. With two charged electrodes, the voltage rise, or decay, is not instantaneous and that finite time for decay is due to instrument and geological effects. In most cases, the effects are very small. However, in some special geological circumstances, the IP effect is a very marked entity. IP is a natural phenomenon generated by inducing a DC-current into the soil, leaving the DC-current on for a short time followed by current shut off. When the current is on, it charges and polarizes the soil, and the IP response of the soil is recorded by measuring the voltage decay after the current is shut off. For use with the resistivity piezocone, this test is carried out when the piezocone is at rest (e.g., during a rod-break or pore pressure dissipation).

The IP-response of the soil is very site specific and the applicability of the IP concept is likely very limited in general practice. However, the technique has been used extensively in mineral exploration for the detection of sulphide minerals. The IP-response can be measured in several ways but is normally characterized by the pulse-transient method whereby voltage decaying curve is utilized (time versus voltage) as shown in Figure 4.16. The more chargeable soils demonstrate slower charge decay. To describe this decaying curve with one value, the chargeability is calculated as:
(A) TYPICAL PROCEDURE FOR PULSE TRANSIENT METHOD. OFF PERIOD ALLOWS HIGH FREQUENCY TRANSIENTS TO DIE DOWN BEFORE SWITCHING ON RECEIVER; DIFFERENCES IN TIME POINTS ARE USUALLY OF THE ORDER 0.5 SECS. AREA UNDER CURVES BETWEEN TIME POINTS IS INTEGRATED AND OFTEN IS NORMALIZED TO $V_0$. SECOND AREA ABOVE CURVE IS SOMETIMES INTEGRATED TO GIVE MORE INFORMATION ON SHAPE OF DECAY CURVE.

(B) ALTERNATIVE APPROACH TO OBTAINING MORE INFORMATION ON SHAPE OF DECAY CURVES; TIME INTERVAL OF INTEGRATION, $\Delta t \approx 0.1$ sec.

Figure 4.16 Concept of Induced Polarization (adapted from Beck, 1981)
\[ M = \frac{1}{V_o} \int_{V_i}^{V_f} V(t) dt \]  

(4.14)

The above parameters are shown schematically in Figure 4.16. More details on the concept of IP and chargeability is available in most introductory geophysical texts (e.g., Telford et al., 1977, Beck, 1982).

Conceived by the Author, the concept of IP measurements with the resistivity piezocone was trialed in the summer of 1996 at the sulphide-rich Sullivan Mine iron tailings area. Kristiansen (1997) utilized this tool and the Sullivan data for his M.A.Sc. thesis and this latter document should be consulted for further details.

4.7 Fast Penetration and Cyclic Piezocone

One of the intriguing aspects of the piezocone is that there are two independent measures of material resistance; tip resistance and sleeve friction. With tip resistance, the analogy of a push-in conical pile allows comparison with bearing capacity formulae through cavity expansion theory. The friction sleeve, however, is more of an in-situ direct shear test. Moreover, the friction sleeve imparts considerable strain on the soil influenced by the piezocone penetration; at least 10’s of percent and likely much more.

It is proposed that the friction sleeve could potentially be an indicator of the high-strain shear strength of the in-situ material although, for mine tailings and most natural soils, the stress path is perpendicular to the preferential plane of sedimentation imbrication. During the research, this concept was developed (as described in Section 9.7.7) in three stages:
• assessing standard $f_s$ data and comparing the values to “expected” strength values;
• assessing $f_s$ data from “fast” penetration testing where the hydraulic pushing rate was increased in increments to a maximum 10 cm/s to see if undrained conditions in silts and sands could be obtained; and
• developing and assessing “cyclic” piezocone testing with both modified hydraulic and data acquisition conditions.

A literature search showed very little discussion on the first item, no specific reference to fast testing re. strength determination and, as far as the Author is aware, the cyclic piezocone testing is an initial concept and effort. Summary results from the cyclic piezocone testing are presented in Section 9.6.7.

One of the problems is that $f_s$ measurements are partially to wholly a measurement of steel-soil frictional resistance versus a soil-soil measure. Using the steel pile analogy, the American Petroleum Institute (API) suggest the steel-soil frictional resistance is roughly $\delta = 20^\circ$ or some $2/3$ of the typical $\phi'_{cv} = 30^\circ$ to $33^\circ$ of the soil-soil equivalent. From this pile analogy, as shown on Figure 4.17, the frictional resistance over soil element $i$ will be as per Equation 4.15.

\[
\begin{align*}
   f_{s_i} &= \sigma' \tan \delta \\
   \text{or} \\
   f_{s_i} &= K\sigma'_v \tan \delta
\end{align*}
\]  

(4.15)
Figure 4.17  Idealized Stress Conditions for Soil Element Adjacent to Piezocone
Although both the absolute stress conditions and stress ratio is unknown after driving, pile installations based on such designs have performed well where the design used initial stress conditions. If we assume a typical value of $K \equiv 0.4$ for most soil-tailings conditions (little stress history), than the estimate of frictional resistance becomes:

$$f_{s_l} = 0.4\sigma'_{vo} \tan \delta$$

(4.16)

where $\delta = 20^\circ$; or

$$f_{s_l} = 0.15\sigma'_{vo}$$

(4.17)

Consequently, according to the above logic, equation 4.17 indicates that the sleeve friction should be a predictable quantity in constant K conditions with only effective stresses being required for $f_s$ quantification. However, the effective stress adjacent to the friction sleeve is not trivially derived. The pore pressure within the soil mass adjacent to the friction sleeve is neither hydrostatic nor equivalent to either $U_2$ or $U_3$ along its entire length. This condition is not a large issue in fully-drained penetration (e.g., coarse sands), but for the majority of mine tailings, partially undrained conditions exist during penetration and complex shear-induced pore pressures are developed.

Two steps were made in the research to create a more constant (analyzable) situation:

1. a ridged friction sleeve, as shown schematically in Figure 4.18, was used to ensure soil-soil frictional response, even at high strains; and
Figure 4.18  Schematic of Modified Friction Sleeve for Cyclic Piezocone Testing
2. the hydraulic pushing arrangement was modified to allow faster, and cyclic, penetration whereby undrained conditions in at least some largely cohesionless tailings could be achieved.

The results of the preliminary efforts with the cyclic piezocone are presented in Section 9.7.7.

4.8 Piezocone Scale Issues

Scale effects on the piezocone, and modifications such as the resistivity piezocone, are important to understand where inhomogeneity and/or anisotropy of soil conditions (e.g., material parameters) can be expected. Except under unique and ideal circumstances, it is the Author’s experience that neither the physical or chemical properties of mine tailings can be described as isotropic or homogeneous over any appreciable volume. For example, a spigotted tailings beach will typically have an anisotropy in hydraulic conductivity such that the horizontal to vertical values are up to two orders of magnitude different (Vick, 1990).

For parameters based upon piezocone tip resistance, it should be understood that the tip is influenced by the state of soil up to approximately 10 diameters below the piezocone tip (or roughly 350 to 360 mm for a standard cone tip). This volume of influence has been shown experimentally (Huntsman, 1985) and analytically, although more effort on modeling piezocone insertion is required. Therefore, for tailings with layerings of less than 50 to 100 mm thick, the lowest strength materials may have their competency overestimated with the reverse being true for the thinner denser and higher strength layers. Robertson and Fear (1995) present an empirical
"thin layer correction" for assessing meaningful layer thickness for liquefaction susceptibility assessments.

Marsland and Quartman (1982) demonstrated the fabric dependency by scale of CPT estimated undrained strength in overconsolidated clays. Fissuring of mine tailings, particularly for finer-grained tailings deposited subaerially, offers a good physical analogy to an overconsolidated clay. Accordingly, where finer-grained tailings have been desiccated on subsequent exposed beaches, the strength of these layers should be computed with caution; particularly when selecting “N” factors in the undrained strength as a function of $q_c/N$ type relationships. Negative pore pressures and “cycling” of wetting/drying tends to create overconsolidation in many finer-grained tailings. This appears to be particularly true for tailings from higher micaceous content ore bodies.

Sleeve friction values are an average over the 150-mm length of the sleeve. Clearly, in layered tailings deposits the values of $f_s$ will alternatively over and underestimate resistance over at least the transition interval for layers of appreciable differences in initial and worked stiffness.

For hydrogeological parameters estimated from the pore pressure dissipation data from U1, U2 or U3 (see Section 10), little research is available to indicate how large the radial or cylindrical cavity of influence actually is in comparison to the idealized analytical half-spaces. As a rough guide, the radius of influence, at least for U1 and U2, is likely similar to that for $q_c$; i.e., about ten cone diameters. It is likely that the radius of influence for U3 is considerably less.
For the bulk resistivity measurements obtained from the RCPTU, Keys (1989) estimates that in soil-fluid systems, the bulk resistivity value is averaged over a quasi-spherical space approximately two times the electrode spacing as shown on Figure 4.19. In Figure 4.19, the electrode spacing, a, appreciates the presence media where \( \rho = \rho_1 \) only when that media is within 2a of the resistivity measurement electrode. Therefore, for the various resistivity modules used in this research, Table 4.6 estimates the approximate penetration of measurement.

Table 4.6 Approximate Radius of Influence for the ISTG Resistivity Modules

<table>
<thead>
<tr>
<th>ISTG RESISTIVITY MODULE</th>
<th>ELECTRODE SPACING (mm)</th>
<th>RADIUS OF MEASUREMENT INFLUENCE (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RES001</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>75</td>
<td>150</td>
</tr>
<tr>
<td>RES002</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>26</td>
<td>52</td>
</tr>
<tr>
<td></td>
<td>77.5</td>
<td>155</td>
</tr>
<tr>
<td>RES003</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>150</td>
<td>300</td>
</tr>
</tbody>
</table>
4.9 Pore Fluid Sampling

4.9.1 Commercial BAT Sampling

As part of the research, the commercially available BAT System (named after the inventor, Bengt-Arne Torstensson, 1984) pore fluid sampling was evaluated. The BAT system consists of a sampling tip that is accessed through sterile evacuated glass sample tubes and a double-ended hypodermic needle set-up. The tube sampler is lowered either by cable or electrical wire depending upon whether a pore fluid sample or a pressure test is being carried out. The equipment is hydraulically pushed with the same equipment used for cone penetration testing.

The BAT sampling tip used by the UBC-ISTG consists of a probe slightly larger in diameter than the resistivity module (e.g., 50-mm versus 44-mm for the friction reducer or for the larger resistivity modules). This sampling tip can be pushed on its own or down the same alignment as the smaller piezocone sounding using drilling rods. Standard AWL flush-joint well-line casing rods work well for the BAT system. A schematic of a typical commercial BAT System is shown in Figure 4.20.

During this research, a comparison was made to relate the accuracies of the two insertion methods (own hole versus expanding a piezocone-sounding hole). Indications from the mine tailings research are that there is no difference in measured results. The US-EPA and other high conformance level groups have adopted BAT technology as appropriate and preferred for many geoenvironmental applications. The attraction of no drill cuttings and the repeatability of the data are cited as the key reasons for this preference. BAT technology has been scrutinized by many investigators and has met with widespread acceptance (e.g. Zemo et al., 1992).
Figure 4.20  Commercial BAT Pore Fluid Sampling Tool
The Author found using the BAT system in conjunction with a resistivity piezocene program provides an excellent screening and focused discrete sampling tandem. The resistivity piezocene log is used to identify depths where groundwater samples are desired based upon the soil behaviour, as estimated by the mechanical channels, and bulk resistivity trends. The BAT is then used by pushing the porous filter element on the BAT probe to a depth within the zone(s) of interest. As noted above, water samples are taken by a wire-line evacuated sample tube, using a needle-punch system that allows purging of the first sample and then in-situ pore water sampling. Sterile glass vials (typically 35 mls or larger) are used for water and gas sample recovery.

After the water sample is retrieved to the ground surface, preliminary chemical tests are conducted on-site and the sample is then stored for further chemical analysis. The field measurements include, at a minimum, measurements of conductivity, temperature, and pH. The samples are stored in 0.1% HCl in sterile containers in a refrigerated environment (less than 4°C). Once enough sampling is carried out at a specific depth, the probe is then pushed to the next depth and the procedure repeated. There is no limit to the number of samples that can be taken at one location.

**4.9.2 KBAT**

A modification of the commercially available BAT System was developed during the research. As described above, the original system consists of a sampling tip that is accessed through sterile evacuated glass sample tubes and a double-ended hypodermic needle set-up. The tube sampler is lowered either by cable or electrical wire depending upon whether a pore fluid sample or a pressure test is being carried out. The modifications made during the research include using a
stainless steel sampling carrier of approximately 120 ml and replacing the hypodermic needle system with Swagelock™ fittings. This latter modification allows much more accurate and feasible sampling in higher total suspended solids (TSS) environments as experienced, for example, during the sampling of metallic mine tailings. The commercial BAT needle-punch system was consistently clogging during test work in mine tailings; a frustrating and practically limiting shortcoming. No such problems were found with the UBC-ISTG KBAT; even in the highest TSS and turbid water environments. Figure 4.21 shows the KBAT water-sampling set-up is also an in-situ hydraulic conductivity system and can be used to estimate hydraulic conductivity. The KBAT conductivity test is carried out by pushing the KBAT-probe with the casing-extensions to the depth of the test and using positive versus negative pressure; i.e., outflow versus inflow. A cylindrical tube is lowered until it reaches the KBAT-probe. The tube is partially filled with inert water and then pressurized. When the tube connects onto the KBAT-probe, the water is forced into the soil. The rate of the outflowing water is recorded, and the data can be interpreted to provide estimates of the hydraulic conductivity of the soil.

More details on the KBAT are presented in Chapters 10 and 11.

4.10 Surface Electromagnetic Methods

Inductive electromagnetic surveying at low and very low frequencies (say below several kHz) has gained recent popularity in the assessment of contaminated sites or for evaluating areas where suspected contamination exists. Long a key element of many geological exploration programs, the engineering community is increasingly embracing surface electromagnetic (EM) methods for several reasons:
Figure 4.21 ISTG KBAT
• the elegant simplicity of the methods;
• the non-intrusive (i.e. no drilling) nature of the methods; and
• the relatively low cost of the methods.

However, this growth in popularity has been hindered by the very large and real limitation that is common to all surficial geophysical techniques; infinite non-unique interpreted solutions are available.

This research attempted to be an initial effort at bringing the surface EM methods from their generally purely qualitative state to whereby quantitative conclusions could be established. To achieve this, the RCPTU has been used to provide subsurface calibration (ground truthing) for the surface values. The only adjustment required working between RCPTU and EM values is an acknowledgment of differences in measurement space, resistivity versus conductivity respectively, and an understanding of representative volume being measured. A net result which will ideally come from advancing this initial research, described in Chapter 12, is to combine RCPTU and EM data to provide accurate three-dimensional screening assessments of subsurface conductive or insulating anomalies.

The theory governing surface EM measurements is analogous to that for the resistivity piezocone. McNeill (1980) provides a fairly complete synopsis of surface EM techniques including the EM-31 and EM-34, which were the tools used in this project.
The EM-31 is a fixed-spacing electrode configuration instrument, which is carried by a single user. A schematic view of the EM-31 and its application electromagnetic field in both the vertical dipole mode (VDM) is shown on Figure 4.22. The EM-31 can also be used in the horizontal dipole mode (HDM). For this research, the use of the VDM was almost exclusively adopted.

The EM-34 is a variable-spacing electrode configuration instrument consisting of two 1 m diameter coils, and is conceptually similar to the EM-31. Both vertical and horizontal dipole mode can be applied at 5, 10, 20, 40 m spacing.

Ground conductivity meters, like the EM-31 and EM-34, actually have a practical limit of measurement. Figure 4.23 shows the approximate limitation of the low induction linearity assumption; approximately 70 m-mhos/m.
Figure 4.22  Induced Current Flow for Surface Electro-magnetics (adapted from McNeill, 1980)
Figure 4.23 Measured Conductivity versus True Conductivity for EM31 (adapted from McNeill, 1980)
4.11 Summary of In-Situ Technologies Used in This Research

For the most part, this research utilized a two-step approach for assessing the thesis of how well piezocone technologies were suited to mine tailings site characterization. This two-step approach combined the standard piezocone data suite and downhole resistivity mapping with follow-up groundwater sampling and appropriate field/laboratory geochemical testing. The continuous record with depth of standard piezocone data and bulk resistivity was obtained with the resistivity piezocone. Geotechnical parameters were largely evaluated from the physical piezocone data. The addition of the resistivity module allowed near-continuous bulk resistivity measurements to be obtained. This resistivity data was then used to determine whether some form(s) of dissolved or free product constituent was present at or above background/baseline values. Background values were established either from on-site experience or from similar geological/geochemical environments. Areas where background values exceeded were then further evaluated with the KBAT groundwater sampling system that recovered samples at specific depths for in-depth field and/or laboratory chemical analyses. This combination of a detailed logging/environmental screening technology and a specific depth groundwater sampling system provided a rapid means of carrying out site characterization at various mine tailings. The research summarized within the remainder of the thesis demonstrates how technically effective the two-step combination is and how it may be expected to perform in a commercial environment.
5. MINE TAILINGS DATABASE

5.1 General

As noted in the introduction, a considerable database has been assembled in the course of the supporting fieldwork for the research. Although there are larger site-specific databases, this assemblage may well represent the most comprehensive piezocone database available specifically related to a variety of mine tailings. The database is largely documented through a series of documents compiled by the Author and issued on behalf of the UBC-ISTG to the appropriate funding agent or minesite. Table 5.1 lists the key references to the database including documents not composed by the Author. With these latter documents, the Author and/or the UBC-ISTG was involved in the procurement of the piezocone contributions to the database.

Table 5.1 Key References to Mine Tailings Research Piezocone Database

<table>
<thead>
<tr>
<th>TITLE</th>
<th>AUTHORSHIP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inco-MENDO 1993</td>
<td>UBC-ISTG (*) (1994e)</td>
</tr>
<tr>
<td>Gibraltar 1994</td>
<td>UBC-ISTG (*) (1994a)</td>
</tr>
<tr>
<td>Gibraltar 1995</td>
<td>UBC-ISTG (*) (1995c)</td>
</tr>
<tr>
<td>Sullivan 1994</td>
<td>UBC-ISTG (*) (1994b)</td>
</tr>
<tr>
<td>Sullivan 1995</td>
<td>UBC-ISTG (*) (1995a)</td>
</tr>
<tr>
<td>Sullivan 1996</td>
<td>UBC-ISTG (*) (1996b)</td>
</tr>
<tr>
<td>Endako 1995</td>
<td>UBC-ISTG (*) (1995b)</td>
</tr>
<tr>
<td>Endako 1996</td>
<td>UBC-ISTG (*) (1996a)</td>
</tr>
<tr>
<td>Trail 1994</td>
<td>UBC-ISTG (*) (1994c)</td>
</tr>
<tr>
<td>SCBC Summary 1997</td>
<td>UBC-ISTG (*) (1997)</td>
</tr>
<tr>
<td>CANLEX Phase I</td>
<td>CANLEX TEAM (**)</td>
</tr>
<tr>
<td>CANLEX Phase II</td>
<td>CANLEX TEAM (**)</td>
</tr>
<tr>
<td>CANLEX Phase III</td>
<td>CANLEX TEAM (**)</td>
</tr>
<tr>
<td>CANLEX Phase IV</td>
<td>CANLEX TEAM (**)</td>
</tr>
</tbody>
</table>

(*) denotes thesis Author as main or sole contributor  
(**) CANLEX documents largely composed by Fear and Robertson, University of Alberta
Figure 5.1 shows the approximate location of each of the main research sites in the database. To demonstrate the breadth of information available in the database, and to adequately substantiate the source of the data utilized in the remainder of the thesis, a summary of the sites used to develop the research database is presented in Appendix I. Appendix I presents site descriptions for the main research sites. There are also a few typical piezocone-sounding traces, with comments on the traces, from one site to allow a general appreciation of the nature of the research work carried out for each site. More detailed information for each site in the database, e.g. piezocone sounding plots and geochemical data, can be obtained from the appropriate references cited in the remainder of the thesis or in Appendix I.

5.2 Main Projects

5.2.1 General

As noted above, the thesis research involved supporting fieldwork from several projects. Of these projects, there were three main contributors to the database which were responsible for more than 90% of the piezocone data acquired for the research.

Following are brief descriptions of the three main projects used in developing the database for this research. Appendix I provides more specific details in each case although the project specific references are recommended for any detailed information not available in this Chapter. For two of the projects, the MENDO-INCO and the SCBC, funding details are summarized in the following text. The funding information is provided to allow other potential researchers to appreciate the scale of the effort involved to achieve the database and thereby assist the planning and execution of any similar future endeavors.
Figure 5.1  Tailings Database Site Locations
5.2.2 MENDO-INCO

The Canadian Centre for Mineral and Energy Technology (CANMET) is the key research and development arm of Natural Resources Canada. Starting in 1987, CANMET initiated the Mine Environment Neutral Drainage (MEND) initiative to coordinate the applied research and development associated with the Acid Rock Drainage (ARD) phenomenon. Several provinces augmented the CANMET-MEND initiative to ensure regional representation and perspective. One of these provinces is Ontario with their initiative, MEND Ontario or MENDO. The UBC-ISTG approached MEND/MENDO and INCO’s Sudbury operations about the possibility of carrying out an initial test of the resistivity piezocone technology in a sulphide tailings environment. Upon acceptance of the Author’s research proposal, MENDO provided partial funding along with INCO Ontario for the proposed comprehensive testing program which was carried out in late 1993. Of the $25,000 in cash contribution, MENDO contributed 60% and INCO Ontario 40%. INCO provided further, and considerable, in-kind support to the research program.

To complete the MENDO-INCO work, which was largely carried out at INCO’s Copper Cliff tailings facility near Sudbury, Ontario, the UBC-ISTG testing vehicle was rail transported. The intent of the program was to demonstrate the capabilities of available resistivity piezocone testing techniques for use in the characterization of materials produced or affected by mining sulphide ores. Specifically, the use of the resistivity piezocone and specialized discrete water sampling techniques were evaluated for their ability to characterize sulphide ore tailings which have undergone oxidation processes.
The main investigation site at INCO was the Central Tailings Area. A total of 13 days were spent carrying out 32 mainly resistivity piezocone soundings for demonstration and research purposes (i.e. in a non-production mode). Several of the sounding locations at the toe of the Pistol Dam were directly adjacent to University of Waterloo sampling piezometer nests. As a consequence of the University of Waterloo installations, independent water chemistry data was of reputable quality available in this area of the minesite. Notwithstanding the research data acquisition capacity, a total of nearly 550 metres (about 1800 feet) of piezocone soundings and 10 BAT water samples were achieved during the INCO program. In addition, one day was spent at Falconbridge Ltd.'s Fault Lake Tailings Area where two resistivity piezocone soundings totaling nearly 65 m were carried out.

Appendix I presents more details regarding the INCO-MENDO project.

5.2.3 SCBC

The Science Council of British Columbia (SCBC), through their Technology B.C. initiative, provided partial funding for a comprehensive applied research program carried out over the period 1994-1997. The concept of the project was to evaluate existing, and develop new, in-situ testing tools and procedures for evaluating the geoenvironmental character of mine tailings. The SCBC project, as it came to be known, was the largest and most comprehensive project for the Author's research in terms of equipment development, funding for technician and summer student support, funding for field work and in the amount of actual field work carried out. Originally approved for two years (1994 and 1995), a third year was added to allow additional
testing and tool development (e.g., IP resistivity piezocone RES003, enhanced geochemical database). A final report was issued in early 1997 as noted in Table 5.1.

Table 5.2 presents the nature of the funding arrangement for the SCBC project. Mr. Jim Robertson, P.Eng., of Placer Dome Inc., acted as the Principal Investigator for the project with the Author acting as the Project Manager.

<table>
<thead>
<tr>
<th>FUNDING PARTICIPANT</th>
<th>CASH CONTRIBUTION</th>
<th>IN-KIND CONTRIBUTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCBC</td>
<td>66,700</td>
<td>project liaison</td>
</tr>
<tr>
<td>Placer Dome Inc.</td>
<td>40,000</td>
<td>corporate support</td>
</tr>
<tr>
<td>Cominco Ltd.</td>
<td>40,000</td>
<td>corporate support</td>
</tr>
<tr>
<td>Klohn-Crippen Consultants Ltd.</td>
<td>-</td>
<td>project accounting, report binding, etc.</td>
</tr>
<tr>
<td>Endako Mines Ltd.</td>
<td>-</td>
<td>fuel, access, office facilities, site data</td>
</tr>
<tr>
<td>Gibraltar Mines Ltd.</td>
<td>-</td>
<td>fuel, access, office facilities, site data</td>
</tr>
<tr>
<td>Cominco, Kimberly Operations</td>
<td>-</td>
<td>access, office facilities, site data</td>
</tr>
<tr>
<td>Cominco, Trail Operations</td>
<td>-</td>
<td>access, office facilities, site data</td>
</tr>
<tr>
<td>UBC-ISTG</td>
<td>-</td>
<td>research vehicle, soft-funded technicians</td>
</tr>
<tr>
<td></td>
<td>146,700</td>
<td>~80,000</td>
</tr>
</tbody>
</table>

Note: 1 Canadian funds

As noted, the information in Table 5.2 is included to show the considerable resources this type of field-based research requires.

During the project’s tenure, quarterly progress reports were submitted to the SCBC for approval. In addition to the technical data reports noted in Table 5.1, ten such reports were assembled and detail the research activity quarter. Should the reader wish to review such information, copies are filed with the SCBC, the Author and the UBC-ISTG.
Field trips to several mine sites in the years 1994, 1995 and 1996 were the main field component of the project each year. However, over 20 field days per year on the local calibration sites were also carried out as part of the SCBC project. The field trips to the various mine sites and their duration are listed in Table 5.3.

**Table 5.3 SCBC Field Program**

<table>
<thead>
<tr>
<th>MINE SITE</th>
<th>1994</th>
<th>1995</th>
<th>1996</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gibraltar</td>
<td>Aug. 29 - Sept. 8</td>
<td>Aug. 23-28</td>
<td>-</td>
</tr>
<tr>
<td>Trail</td>
<td>Oct. 17 - 20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sullivan</td>
<td>Oct. 4 - 14</td>
<td>July 18 - 26</td>
<td>July 30 - Aug. 9</td>
</tr>
<tr>
<td>Endako</td>
<td>-</td>
<td>Aug. 15 - 22</td>
<td>July 3 - 12</td>
</tr>
</tbody>
</table>

The majority of the SCBC piezocone field program was carried out by utilizing the UBC-ISTG in-situ testing vehicle. Water samples and soil samples were also collected, and laboratory tests conducted on these samples. Surface geophysical methods described in Chapter 4 were also heavily utilized during the project. Project statistics for the testing carried out are listed in Table 5.4 and 5.5.

**Table 5.4 SCBC Project Statistics - Piezocone Technology**

<table>
<thead>
<tr>
<th>PIEZOCONE TECHNOLOGY</th>
<th>SPECIFICS</th>
</tr>
</thead>
<tbody>
<tr>
<td>RCPTU</td>
<td>113 soundings = 1800 m</td>
</tr>
<tr>
<td>Piezocone Dissipations</td>
<td>370 tests to minimum t50</td>
</tr>
<tr>
<td>Cyclic</td>
<td>3 tests</td>
</tr>
<tr>
<td>Seismic</td>
<td>76 tests</td>
</tr>
<tr>
<td>IP</td>
<td>31 tests</td>
</tr>
</tbody>
</table>
Table 5.5  SCBC Project Statistics - Supporting Technologies

<table>
<thead>
<tr>
<th>SUPPORTING TECHNOLOGY</th>
<th>SPECIFICS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geochemical pore water samples from BAT/KBAT</td>
<td>70 samples</td>
</tr>
<tr>
<td>(laboratory tested)</td>
<td></td>
</tr>
<tr>
<td>Soil samples</td>
<td>42 samples</td>
</tr>
<tr>
<td>Em 31 &amp; Em 34 surveys</td>
<td>27 km</td>
</tr>
<tr>
<td>KBAT (outflow)</td>
<td>5 tests</td>
</tr>
<tr>
<td>Slug tests</td>
<td>5 tests</td>
</tr>
<tr>
<td>Self-boring pressuremeter</td>
<td>17 tests</td>
</tr>
<tr>
<td>Stream water conductivity survey</td>
<td>60 readings</td>
</tr>
<tr>
<td>Laboratory Triaxial Testing</td>
<td>4 samples</td>
</tr>
</tbody>
</table>

The UBC-ISTG in-situ testing vehicle was used to collect the data for all of the tests except for the soil sampling, slug tests and EM 31 and EM 34 surficial geophysics.

5.2.4 CANLEX

The Canadian geotechnical engineering community, under the leadership of Dr. Peter Robertson and his colleagues at the University of Alberta, initiated a major study investigating the liquefaction of sand entitled the Canadian Liquefaction Experiment (CANLEX). The CANLEX Project was a collaborative effort of industry, engineering consultants and university participants, with the support of the Natural Sciences and Engineering Research Council of Canada (NSERC). The study examined sand characterization for dynamic and static liquefaction. The project commenced in 1993 and was completed in 1998. The industry participants in the project included Syncrude Canada Ltd., Suncor Inc., B.C. Hydro, Hydro Québec, Highland Valley Copper and Kennecott Corporation. The geotechnical consultants participating in the CANLEX Project were EBA Engineering Consultants Ltd., Klohn-Crippen Consultants Ltd., AGRA Earth & Environmental Engineering Ltd., Golder Associates Ltd., and Thurber Engineering Ltd. The study also collaborated with faculty and students from the University of Alberta, University of British Columbia, Carleton University, Université de Laval and the Université de Sherbrooke.
The main objectives of the project included:

- developing test sites to study sand characterization;
- developing economical undisturbed sampling techniques in which in-situ freezing is seen as the most promising technique; and
- evolving a greater understanding of soil liquefaction.

The project was divided into different phases, with each Phase extending for approximately one year with a specific set of objectives. The research for this thesis benefited from all four phases of the project:

- Phase I - characterization work at Syncrude Canada Ltd.’s (SCL) Mildred Lake Settling Basin (MLSB), near Fort McMurray, Alberta.
- Phase II - characterization efforts on the Fraser River Delta of British Columbia.
- Phase III - static liquefaction triggering event at SCL’s J-Pit.
- Phase IV - site characterization at Highland Valley Copper’s (HVC) tailings facility near Logan Lake, British Columbia.

Project Phases I, III and IV involved mine tailings whereas Phase II doubled as this research’s local calibration sites. The Author was heavily involved in the field programs for Phases I and II and remained close to the project in the later phases. The CANLEX database offers an excellent contribution to the physical characterization portion of the overall tailings database of this thesis.
5.3 Other Contributions to Database

There were a number of other sources of data used in forming the overall database for this research. In many cases, information was reviewed and not used for lack of certainty of data procurement quality or similar concerns. However, there were specific instances where the integrity of the data was judged to be sound.

Sources of information included personal communication with other researchers, the Author's own project files, literature and from industry representatives (e.g. a technical representative from a given minesite). Wherever this data is utilized in forming the conclusions offered by this thesis, the specific reference to the source of the data is included at that point in the thesis.
6. **SOIL BEHAVIOR TYPE**

6.1 **Overview**

Soil behaviour type discrimination using piezocone data is likely the most widely used aspect of piezocone technology. As described in this chapter, there has been a progression in available methodologies over the past decade which has seen gradual acceptance of stress-normalized soil type classification. The Author's original contributions include Jefferies and Davies (1991) which was completed prior to this research. The concept of material index, \( I_c \), was initially proposed early in the research by the Author in Jefferies and Davies (1993) and now is gaining use in piezocone practice (Lunne et al., 1997). The linear relationship between \( I_c \) and fines content presented is an additional original contribution that should prove useful to practitioners.

6.2 **Perspective**

One of the earliest uses of cone penetration data was for estimating the stratigraphic sequences of the soil sites being investigated. The addition of the pore pressure sensing element to the cone penetration test added a tremendous amount to these interpretive efforts; particularly in finer-grained soils. As noted in Chapter 3, mine tailings are most often finer-grained soils with typical gradations having a minimum of 40% to 60% finer than 74 \( \mu \)m.

Available literature tends to refer to the piezocone as a stratigraphic logging tool. This perspective can present a perception problem for the technology. Engineers tend to find an error in the interpreted stratigraphic section versus exact soil descriptions from samples and then cast doubt at all piezocone data. It is then reasonable that these same individuals become dubious regarding more specific parameter estimations from the piezocone noting that if it makes errors
in its main application (stratigraphic logging) then how good could it be for more complex estimations? The Author suggests that piezocone technology should be more fairly and accurately emphasized as tools that provide an indication of soil behaviour type versus “black box” soil loggers. Often, soil behaviour type and the actual stratigraphic unit are equivalent. However, particularly in “special soils” such as mine tailings, where, for example, fine-grained does not necessarily mean cohesive, the equivalence is less likely.

The Author suggests that the logging capabilities of the piezocone be judged on a question of what is more important:

- one, knowing the correct geologic/stratigraphic name of a soil more important or,
- two, knowing how it will behave under the prevailing in-situ conditions?

At least for the characterization of mine tailings in all practical situations, the latter is seemingly what is of importance and therefore required. The piezocone, when properly interpreted, does an excellent job in describing the equivalent soil behaviour type of the material being tested (Lunne et al., 1997). This was also found to be true for mine tailings as is shown in this chapter. As noted, what should be avoided is the view that the piezocone is a “black box” that downhole logs the soil profile much in the same manner as a geologist logs soil or rock core.

6.3 Existing Methods for Estimating Soil Behaviour Type

Estimation of soil type behaviour with the piezocone is largely empirical. As the cone is advanced, the forces measured by the tip and the friction sleeve vary with material properties of
the soil being penetrated. Very simplistically, sandier soils have higher tip stress measurements relative to their measurement values of sleeve friction than is found for finer-grained sediments. The excess pore pressure ($\Delta u$) measured during penetration provides an indication of soil drainage. Traditionally, the best soil-type piezocone interpretation methods combine the tip and sleeve interpretation with some type of pore pressure interpretation.

The first “modern” soil type interpretation methods did not take pore pressure measurements into account. Nonetheless, charts such as Figure 6.1 provided good soil typing and often stratigraphic equivalence. The next “step” in interpretation, which was really a modification of Figure 6.1, were multi-zoned efforts such as shown on Figure 6.2. The refinement from Figure 6.1 to 6.2, which was fully justified to the experienced user of piezocone data, created some confusion in general engineering use as the increase in “zones” resulted in even less stratigraphic equivalency to interpreted soil type for “non-standard soils” or soils with higher in-situ stress levels. Even more complicated charts than Figure 6.2 have been proposed which, as per Figure 6.2, offered the experienced piezocone user excellent tools but tended to be excessive for general use.

Regardless of complexity arguments, neither soil type classification shown in Figures 6.1 or 6.2, or similar efforts, are suggested for mine tailings. The problem with such classification systems is the inherent problem of non-stress normalization. As noted in Chapter 4, there is a need for a unified interpretation of soils, including mine tailings, which eliminates the artificial soil type changes brought about by the influence of stress level on the measured quantities with the piezocone. For example, Figure 6.3 presents one of the earlier CPT interpretation correlations used with the modern electric cones; the estimation of in-situ relative density. For illustration,
Figure 6.1 Traditional CPT Soil Behaviour Interpretation Chart (adapted from Robertson and Campanella, 1983)
Figure 6.2  Multi-Zone Soil Behaviour Interpretation Chart (adapted from Campanella and Robertson, 1984)
Figure 6.3 Relative Density Relationships for the CPT (adapted from Robertson and Campanella, 1983)

1. SCHMERTMANN (1976) HILTON MINES SAND - HIGH COMPRESSIBILITY
2. BALDI et al. (1982) TICINO SAND - MODERATE COMPRESSIBILITY
3. VILLET & MITCHELL (1981) MONTERY SAND - LOW COMPRESSIBILITY
assume a dense deposit of uniform sand tailings with constant 80% relative density ($D_r$) with depth. This idealized material is located within a tailings impoundment which has a confining dam 100 metres in height. There is a phreatic surface within this structure at a depth of 50 metres. In this fictitious example, the effective vertical stress at 2 metres depth, assuming, say, $y' = 18 \text{kN/m}^3$, would be 36 kPa or roughly 0.4 bar. At 25 metres depth, the effective vertical stress would be 450 kPa (4.5 bar) and at 90 metres, 1520 kPa (15 bar). Figure 6.3 does not provide extension beyond 5 bar but the back-predicted $q_c$ value for the 2 m and 25 m deep locations would be about 80 bar and 400 bar, respectively (if this example had used $D_r = 40\%$, the difference would still be appreciable but less). The dense tailings in this example are indeed a clean sandy material with a friction ratio of about 0.4%. Use of Figure 6.1 would imply that the 2 and 25 m depth samples are “Sands” but Figure 6.2, the more refined interpretation chart, would suggest the increase in depth results in a material crossing from Zone 8 (Sand to Silty Sand) to Zone 10 (Gravely Sand to Sand) for the same material.

A real example can also readily show this stress-level effect. The Sullivan Mine site Old Iron Pond showed tip resistance values near the surface (to 4 metres) of about 20 bar which increased to nearly 100 bar at the base of the tailings at 15 to 20 metres. This material had a relatively constant friction ratio of 0.7% to 0.8%. Moreover, the in-situ density for the material was found to be relatively constant with depth (SCBC, 1994b, 1995a and 1996b); i.e. the material had relatively similar material type and condition with depth. However, Figure 6.1 would indicate that this tailings deposit is a Silty Sand at shallow depth and becoming Sand with depth.
The need to stress normalize piezocone measurements is important where overburden stress exceeds about 150 to 200 kPa (roughly 10 to 20 metres depth, depending upon saturation levels and hydraulic gradients). Stress normalization of measurements, such as outlined in Chapter 4, prior to parameter evaluation is slowly becoming more accepted. It has not, however, become viewed as state-of-practice as the Author predicted at the beginning of the 1990's (e.g., Jefferies and Davies, 1991). Reasons for this slow development include lack of a normalization standard, limited literature emphasizing the importance of stress normalization and limited literature or research on higher stress-level sites such as large tailings dams. The body of literature from the CANLEX project and similar high-level research projects will assist in refocusing the geotechnical community on this important issue.

Robertson (1990) offered a combined stress normalization chart that not only made use of Q and F, but added a companion chart with $B_q$. His work was a major advance in soil behaviour type interpretation under many conditions. The Q and F chart by Robertson (1990) is shown on Figure 6.4. Notwithstanding this advance, applying the Robertson charts in mine tailings and other hydraulic fills can be problematic (e.g., as demonstrated by Jefferies and Davies, 1991). This is especially the case with finer-grained, but cohesionless and brittle, soils such as many mine tailings. Moreover, the chart in Figure 6.4 tends to misinterpret the nature of consolidation stress history, which is readily obtainable from the dimensionless suite of Q, F and $B_q$.

Consequently, at the outset to this research, a soil behaviour interpretive format was not available for piezocone data that would work with hydraulic fill mine tailings. As noted above, soil behaviour type has been adequately estimated from piezocone data for many soils.
NORMALIZED FRICTION RATIO, \( F = \frac{f_s}{q_T - \sigma_{vo}} \times 100\% \)

NOTES:
1. SENSITIVE FINE GRAINED
2. ORGANIC SOILS - PEATS
3. CLAYS - CLAY TO SILTY CLAY
4. SILT MIXTURES - CLAYEY SILT TO SILTY CLAY
5. SAND MIXTURES - SILTY SAND TO SANDY SILT
6. SANDS - CLEAN SAND TO SILTY SAND
7. GRAVELY SAND TO SAND
8. VERY STIFF SAND TO CLAYEY* SAND
9. VERY STIFF FINE GRAINED*

*HEAVILY OVERCONSOLIDATED OR CEMENTED

Figure 6.4 Normalized Piezocone Interpretation Chart (after Robertson, 1990)
Also as discussed above and earlier in this thesis, the influence of drainage on penetration response can be incorporated with a piezocone to enhance interpretation. From the same basics available to previous interpretation methods, a revised interpretation chart for mine tailings is suggested. The suggested chart starts with using the drainage data provided by a piezocone sounding being incorporated into the soil classification scheme using the grouping $Q(1 - B_q)$ which is similar, and a simplification to Equation 4.9 developed in Chapter 4. This grouping was previously, and relatively simultaneously, proposed for unification of piezocone data by Houlsby (1988) and Been et al. (1988) although not previously applied to an interpretation chart. The suggested soil behaviour type relationship for mine tailings can be considered as a modification of Robertson (1990) using the revised grouping of normalized parameters presented in Chapter 4. This modified interpretation chart is presented in Figure 6.5 and is a modification of the Author's initial chart in Jefferies and Davies (1993). The concept of incorporating pore pressure data directly into the $Q(1 - B_q)$ grouping is to expand the interpretation range in finer soils while leaving the interpretation in sands unchanged; a characteristic felt important for mine tailings work.

The following sections illustrate the general success in using the basis of Figure 6.5 for soil behaviour type interpretation of mine tailings.

6.4 Concept of Material Index

As noted above, with the unified classification system per Figure 6.5, a general basis for assessing the soil behaviour type of tailings is proposed. The credibility of this basis is assessed using the database of the mine tailings evaluated during this research. As a reasonable manner
with which to carry out this evaluation, a new parameter for piezocone soil behaviour type indexing is postulated. The concept is based upon the interpretive chart in Figure 6.5 which returns piezocone interpretation to the simplicity of Figure 6.1 with the full advantages of stress normalization and the incorporation of soil drainage conditions.

If the vertical and horizontal scales are distorted by using differing length scales, the boundaries between soil behaviour type zones in Figure 6.5 can be approximated as concentric circles, as shown on Figure 6.6. Within this scaling manipulation, the soil behaviour type boundaries can be characterized by circle radius. To take this one step further, the radius to any combination of \( Q(1 - B_0) \) and \( F \) may be used as a soil behaviour type index. This index, denoted \( I_c \), is defined as:

\[
I_c = \sqrt{3 - \log\left[ Q (1 - B_0) \right]^2 + [1.5 + 1.3 (\log F)]^2}
\]  

In Equation 6.1, \( Q(1 - B_0) \) is dimensionless and \( F \) is in its usual percentage format. As shown on Figure 6.6, the mapping is used to obtain a plot with concentric circles, and the center of the circles is \( \log(Q) = 3, \log(F) = -1.5 \). The logarithms used are base 10. Within this proposed classification of soil behaviour types, values of \( I_c \) and the corresponding behaviour type are summarized in Table 6.1.

**Table 6.1  Soil Behaviour Type from Material Index \( I_c \)**

<table>
<thead>
<tr>
<th>MATERIAL INDEX ( I_c )</th>
<th>ZONE</th>
<th>SOIL BEHAVIOUR TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>( I_c &lt; 1.80 )</td>
<td>6</td>
<td>Clean Sand to Silty Sand</td>
</tr>
<tr>
<td>1.80 &lt; ( I_c &lt; 2.40 )</td>
<td>5</td>
<td>Sand Mixtures - Silty Sand to Sandy Silt</td>
</tr>
<tr>
<td>2.40 &lt; ( I_c &lt; 2.76 )</td>
<td>4</td>
<td>Silt Mixtures - Clayey Silt to Silty Clay</td>
</tr>
<tr>
<td>2.76 &lt; ( I_c &lt; 3.22 )</td>
<td>3</td>
<td>Clays - Clay to Silty Clay</td>
</tr>
<tr>
<td>3.22 &lt; ( I_c )</td>
<td>2</td>
<td>Organic Soils - Peats</td>
</tr>
</tbody>
</table>
Figure 6.5  Unified Piezocone Soil Behaviour Chart
ZONE | SOIL BEHAVIOR TYPE  
---|---
6 | CLEAN SAND TO SILTY SAND
5 | SAND MIXTURES - SILTY SAND TO SANDY SILT
4 | SILT MIXTURES - CLAYEY SILT TO SILTY CLAY
3 | CLAYS - CLAY TO SILTY CLAY
2 | ORGANIC SOILS - PEAT

Figure 6.6 Unified Piezocone Chart with Concentric Zones (adapted from Jefferies and Davies, 1993)
6.5 Evaluation of Material Index

With the concept of material index established, the algorithm presented by equation 6.1 was evaluated using the tailings piezocone database. As predicted, the problems with non-normalized classification schemes were significantly reduced. For example, the Endako Mine tailings are very uniform, although slightly coarser-grained with depth (grinding circuit alterations over the 30+ year mine life), at both tailings impoundments. A number of ~40 to 50 m soundings were carried out at both Pond No. 1 and Pond No. 2. The values of $F$ were anywhere from about 0.35 to about 0.85 with the mean, $\bar{F} = 0.619$ with a standard deviation of 0.291. Figures 6.7 to 6.9, inclusive, show how the values of $I_c$ are consistently trending from about 1.5 to just over 1.75 which befits the actual nature of the tailings which is conventionally described as clean sand to silty sand with a $D_{50} = 0.21$ mm (4 samples) and a <74 \mu m content of between 11% and 20%.

The consistency of the behaviour type classification over the extensive depth (stress level) range at Endako was typical of application of the algorithm for the tailings database.

As another example, a typical $I_c$ profile from the cycloned Gibraltar tailings dam is shown in Figure 6.10. Sounding GB-T9409 was carried out slightly upstream (beachward) of the dam crest. The main dam “sand core” is readily apparent below about 7.5 m ($I_c \approx 1.56$) with the upper beached area showing as a gradual coarsening gradation from $I_c \approx 2.35$ to the lower values. Figure 3.2 showed a schematic of an idealized centre-line cycloned sand dam. Gibraltar’s dam is close to this configuration but has a very slight downstream trend to the crest which was readily evident on the interpreted piezocone sounding via inspection of the $I_c$ profile. Figure 6.10 is a simple example of how the as-built conditions of a tailings dam can be compared to the design
Figure 6.7 \( I_c \) from ENDK9622 (Endako Pond No. 2)
Figure 6.8 $I_c$ from ENDK9619 (Endako Pond No. 2)
Figure 6.9  $I_c$ from ENDK9603 (Endako Pond No. 1)
Figure 6.10  $I_c$ from BG-T9409 (Gibraltar Pond/Dam)
assumptions. In the case of the Gibraltar dam, the subsurface beach geometry confirms the presence of a coarse-sand “core” to the dam underneath the beached tailings which also confirms the slight downstream direction to the core per the design.

Table 6.2 presents a summary of material index for the various types of tailings available to the piezocone tailings database. The fines content noted in Table 6.2 were obtained from gradation tests on tailings obtained from boreholes adjacent to the piezocone soundings.

Table 6.2 Summary of Material Indices for a Range of Tailings Materials

<table>
<thead>
<tr>
<th>TAILINGS</th>
<th>$\epsilon_c$</th>
<th>SOIL BEHAVIOUR TYPE (from Table 6.1)</th>
<th>ACTUAL TAILINGS FINES CONTENT (% &lt;74 $\mu m$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alaskan Tailings</td>
<td>2.81</td>
<td>clay to silty clay</td>
<td>32</td>
</tr>
<tr>
<td>Endako Beach</td>
<td>1.84</td>
<td>silty sand to sandy silt</td>
<td>20</td>
</tr>
<tr>
<td>Endako Dyke</td>
<td>1.60</td>
<td>clean sand to silty sand</td>
<td>11</td>
</tr>
<tr>
<td>Gibraltar Dam</td>
<td>1.56</td>
<td>clean sand to silty sand</td>
<td>11</td>
</tr>
<tr>
<td>Gibraltar Pond</td>
<td>2.36</td>
<td>silty sand to sandy silt</td>
<td>60</td>
</tr>
<tr>
<td>Green’s Creek Pond</td>
<td>3.20</td>
<td>clay to silty clay</td>
<td>90</td>
</tr>
<tr>
<td>HVC Highmont Shell</td>
<td>2.01</td>
<td>silty sand to sandy silt</td>
<td>20</td>
</tr>
<tr>
<td>HVC L-L Shell</td>
<td>1.72</td>
<td>clean sand to silty sand</td>
<td>16</td>
</tr>
<tr>
<td>HVC Trojan</td>
<td>1.68</td>
<td>clean sand to silty sand</td>
<td>12</td>
</tr>
<tr>
<td>²Kidd II (CANLEX target)</td>
<td>1.56</td>
<td>clean sand to silty sand</td>
<td>9</td>
</tr>
<tr>
<td>KUC Shell</td>
<td>2.25</td>
<td>silty sand to sandy silt</td>
<td>55</td>
</tr>
<tr>
<td>KUC Beach</td>
<td>3.45</td>
<td>organic soils</td>
<td>95</td>
</tr>
<tr>
<td>²Massey South (CANLEX target)</td>
<td>1.58</td>
<td>clean sand to silty sand</td>
<td>9</td>
</tr>
<tr>
<td>Myra Falls Dyke</td>
<td>2.39</td>
<td>silty sand to sandy silt</td>
<td>60</td>
</tr>
<tr>
<td>Myra Falls Pond</td>
<td>3.20</td>
<td>clay to silty clay</td>
<td>90</td>
</tr>
<tr>
<td>SCL J-Pit.</td>
<td>2.07</td>
<td>silty sand to sandy silt</td>
<td>31</td>
</tr>
<tr>
<td>SCL Mildred Lake Dyke</td>
<td>1.24</td>
<td>clean sand to silty sand</td>
<td>3</td>
</tr>
<tr>
<td>SCL SWSS BBW</td>
<td>2.22</td>
<td>silty sand to sandy silt</td>
<td>40</td>
</tr>
<tr>
<td>Sullivan Gypsum Pond</td>
<td>3.23</td>
<td>organic soils</td>
<td>70</td>
</tr>
<tr>
<td>Sullivan Calcine Pond</td>
<td>2.52</td>
<td>clayey silt to silty clay</td>
<td>65</td>
</tr>
<tr>
<td>Sullivan Gypsum Dyke</td>
<td>2.75</td>
<td>clay silt to silty clay</td>
<td>60</td>
</tr>
<tr>
<td>Sullivan Old Iron Pond</td>
<td>3.21</td>
<td>clay to silty clay</td>
<td>60</td>
</tr>
<tr>
<td>Sullivan Old Iron Pond</td>
<td>2.78</td>
<td>clay to silty clay</td>
<td>78</td>
</tr>
<tr>
<td>Sullivan SW Limb Dyke</td>
<td>3.10</td>
<td>clay to silty clay</td>
<td>85</td>
</tr>
<tr>
<td>Trail &quot;Glory Hole&quot;</td>
<td>1.39</td>
<td>clean sand to silty sand</td>
<td>2</td>
</tr>
</tbody>
</table>

Notes: ¹ $\epsilon_c$ averaged over depth zone of interest.
² Calibration research site.
It is proposed that the use of $I_c$ is a very simple, easily automated, manner with which to estimate soil behaviour type for mine tailings. As will be further demonstrated in Chapter 7 (in-situ material state), the interpretation pairing of $Q(1 - B_q)$ and $F$ appears well-suited to characterizing more comprehensive aspects of the character of a variety of mine tailings over a considerable range of in-situ stress conditions.

6.6 Fines Content

The amount and nature of the "fines" content of a given soil often control the mechanical and hydraulic behaviour of soils. The traditional definition of fines content is the percentage of material passing a No. 200 screen mesh that has an equivalent-limiting diameter of 74 μm. Estimates of fines content are routinely required in tailings engineering. For example, the most commonly used liquefaction assessment procedures required an estimate of fines content. Traditional SPT methods allow the fines content to be obtained from laboratory testing the split-spoon samples. However, with piezocone technology such physical sampling is not available.

Methods for estimating fines content from piezocone data are few and the practice has not been common. However, there have been some proposed methods which can be grouped into those using either:

1. friction ratio;
2. tip resistance alone; or
3. rate of pore pressure dissipation.
Pore pressure dissipation methods have not been successful as, most notably in non-cohesive fines (i.e., mine tailings), dissipation rates have been rapid in comparison to the rate of commercial data acquisition. Moreover, if the soil (tailings) are even partially dilatant, accurate dissipation information can be difficult to obtain.

For tip resistance alone, there have been some attempts with reasonable but only site-specific success. Most recently, Steedman (1997) suggested a method to carry out textural modeling of copper tailings using the piezocone tip resistance in isolation. Figure 6.11 shows the data and suggested relationship developed by Steedman. There is significant scatter in this data set which is from a single mine site. This scatter is largely a function of limiting the assessment of texture (fines content) to Q and ignoring F. The data set and relationship presented by Steedman (1997) in Figure 6.11 can be reassessed using the $I_c$ concept. Figure 6.12 presents a plot of Q versus F for the data from the copper mine evaluated. Using the concept of $I_c$, the $R^2$ value of scatter in terms of $I_c$ versus fines content improved to roughly 0.9 (using Figure 6.15) from the value of approximately 0.6 from Steedman's tip alone relationship.

Of the friction ratio methods, the relationship be Suzuki et al. (1995) has likely received the most attention and is shown in Figure 6.13. Friction ratio methods alone ignore density variations in the material which can lead to excessive scatter in the data.
Figure 6.11 Copper Tailings Tip Resistance versus Gradation (adapted from Steedman, 1997)
Figure 6.12  Normalized CPT Data from Copper Tailings (adapted from Steedman, 1997)
Figure 6.13  Estimating Fines Content from Friction Ratio (%) (adapted from Suzuki et al. 1995)
From the tailings database established by this research, a suggested alternative to the existing methods is presented based upon material index, \( I_c \). Using the data from Table 6.2, Figures 6.14 and 6.15 can be developed. Although the tailings database is not as extensive as it could be, the relationships in Figures 6.14 and 6.15 are considered an excellent initial effort for practical use.

The reason for the linear and power series plots for the same data was to see if the Suzuki et al. (1995) form was usable or whether a simpler linear relationship existed. As \( I_c \) is truly a measure of behaviour type, it is not unreasonable to expect a linear relationship with fines content. And indeed, Figure 6.15 appears to have a decent \( R^2 \) value (~0.9) and few outliers.

For screening purposes, therefore, an estimate of fines content in mine tailings can be obtained from reduced piezocone data, per Chapter 4 and Equation 6.1, from equations 6.2, 6.3 and 6.4.

\[
\text{For } I_c \leq 1.3, \quad FC(\%) = 0 \quad (6.2)
\]

\[
\text{For } 1.3 < I_c < 3.65, \quad FC(\%) = 42.4(I_c) - 54.9 \quad (6.3)
\]

\[
\text{For } I_c \geq 3.65, \quad FC(\%) = 100 \quad (6.4)
\]

The above relationships are not suggested for application to fibrous peat deposits or materials substantially outside of the material types presented in Table 6.2. Application of the concept to natural sediments beyond the Fraser River Delta CANLEX sites has not been rigorously tested by this research.
FINES CONTENT (%) = EXP (1.41671\*lc)*1.08234
NUMBER OF DATA POINTS USED=25
COEF OF DETERMINATION, R-SQUARED = 0.796363

Figure 6.14  Estimating Fines Content from Ic - Power Relationship
FINES CONTENT (%) = (42.4179 \cdot I_c) - 54.8574

NUMBER OF DATA POINTS USED = 25

COEF OF DETERMINATION, R-SQUARED = 0.873205

Figure 6.15  Estimating Fines content from $I_c$ - Linear Relationship
7. IN-SITU STATE

7.1 Overview

The in-situ “state” of tailings is the combined stress and density of the tailings at a given location in a storage facility. The concept of a state parameter, developed by others for mainly natural soils (Been and Jefferies, 1985), is extended herein to include mine tailings. The Author and his colleagues (Plewes et al., 1992) presented an initial relationship between piezocone parameters and in-situ state which include material compressibility, a parameter neglected in the original Been and Jefferies (1985) work.

The current research builds upon the Plewes et al. (1992) contribution by developing a tailings specific method of estimating material state from piezocone data. Included in this original work on tailings is a method for estimating material compressibility with the material index introduced in Chapter 6.

The mechanical behaviour of mine tailings is a function of state. The ability to estimate in-situ state from the piezocone is therefore a suggested manner with which to develop engineering parameters from piezocone data. This thesis presents several relationships between piezocone derived state and key engineering parameters of mine tailings.

7.2 Background

The mechanical properties of soils, including mine tailings, can be fully established if three things are known:
- the in-situ state of stress;
- the in-situ state of density; and
- the governing constitutive relationship for the soil material in question.

The piezocone can provide input parameters to constitutive models but cannot establish specific stress-strain relations. However, the pair of stress and density, termed material "state" when both shear and normal stresses can be known (i.e., perpendicular stresses are known), can be estimated from piezocone data. Material state can be defined by a "state parameter" as described below.

A simple procedure is proposed for estimating the in-situ state of mine tailings using the idealized critical state soil model. This concept of this procedure was originally developed by the Author and his colleagues at the outset of this research (e.g., Plewes et al., 1992) for soil deposits in general. As the tailings research progressed, it has been specifically applied to better fit the tailings database. The procedure incorporates the three basic piezocone measurements of tip resistance, sleeve friction and pore pressure response.

The existence of a range of possible densities for tailings requires a normalizing parameter to reduce the wide range of possible soil behaviours into a manageable framework. Early examples of attempts at meeting this need are the concept of relative density for sands and that of liquidity index for clays. In each case, there is an underlying assumption that all soils with a common index value will exhibit similar constitutive behaviour.
Been and Jefferies (1985) suggested that a simple index, the state parameter $\psi$, would provide an index parameter for practical geotechnical engineering that was nevertheless anchored to an applied mechanical understanding of soil. The state parameter is simply defined as the difference between the current void ratio ($e$) and the critical void ratio ($e_c$) at the same stress level as per Equation 7.1.

$$\psi = e - e_c$$  \hspace{1cm} (7.1)

In two-dimensional space, the state parameter is defined schematically in Figure 7.1. In actual three-dimensional space, the relationship between void ratio, shear and normal stress (e.g. state) is defined by a point within a permissible "volume" of stress and void ratio for a given soil. Schematically, Figure 7.2 shows this idealized volume.

For the particular condition of pure shear, $\psi$ defines the volumetric strain potential which, at least theoretically, is independent of material gradation and mineralogy. At the same time, $\psi$ is a reflection of material gradation, mineralogy, placement fabric etc. Been and Jefferies (1985) and Been et al. (1988) summarized triaxial data on more than twenty sands and showed that peak friction angle and peak dilatancy were strong functions of $\psi$ alone. However, as described below, the original formulation did not capture enough of the true nature of soils, particularly silty and clayey soils, due to a lack of inclusion of material compressibility. Nonetheless, early encouragement from the initial state parameter literature suggested that $\psi$ was at least a reasonable start at an index parameter for developing a rational understanding of soil behaviour from the state of stress and density.
Figure 7.1  Simplified Two-Dimensional Soil State Diagram
Figure 7.2  Simplified Three-Dimensional Soil State Diagram
The definition of $\psi$ is straightforward and the concept is readily applied in laboratory situations where the critical state line (CSL) has been determined and the initial density and stress conditions of individual tests are readily measurable. Application of the concept to actual in-situ conditions requires additional steps as the actual in-situ density is difficult to measure in cohesionless silts and sands, e.g., the majority of mine tailings. Recent advancements in obtaining accurate measures of in-situ density using specialized sampling methods and nuclear density logging as described by Plewes et al., (1988, 1991), and soil freezing work (e.g., Hoffman, 1997) show promise, but are not yet common practice.

The method of determining the state parameter of cohesionless soils from piezocone data was originally developed from the existing database of calibration chamber tests (Been et al., 1987). Chamber tests provide piezocone penetration resistances in terms of soil density and confining stress. Measurement of the CSL for the respective sands used in the chamber testing allowed the data to be transformed into a piezocone-state parameter relationship of the form:

$$Q_p = k \exp(-m \psi) = \frac{q_l - \sigma_o}{\sigma_o'}$$

(7.2)

where $k$ and $m$ are constants which differ for sands of various composition. Note that Been and Jefferies (1985, 1993) prefer mean stresses which are difficult to determine from piezocone data alone. Been et al., (1987) suggest that the constants $k$ and $m$ are each a direct function of only the critical state parameter $\lambda$. However, on purely dimensional grounds and as well from conventional solutions for cavity expansion in a frictional material (Vesic, 1972), it is possible that $k$ and $m$ could
be functions of both \( \lambda \) and a dimensionless coefficient \( G/(q_t - \sigma_o) \) where \( G \) is the shear modulus of the soil.

Although the work of Been et al., (1987) and Been and Jefferies (1993) provides a way of interpreting piezocone data for different sands, tailings contain appreciably greater silt-sized material than the sands tested. The state parameter method for the piezocone interpretation was generalized to include sandy silts and silts using the observation of good correlation between piezocone penetration resistance and overconsolidation ratio of clays (Wroth, 1988; Sills et al., 1988; Crooks et al., 1988). Extending the concept of Crooks et al. (1998), overconsolidation ratio (OCR) can be approximately related to \( \psi \) for clays as:

\[
\psi/\lambda = \log(r) - \Lambda \log(R)
\]

(7.3)

where \( R \) is the isotropic OCR, \( r \) is the spacing between the CSL and normally consolidated line (NCL), and \( \Lambda \) is the critical state plastic hardening ratio as described in Plewes et al. (1992). For most purposes, \( r \) may be treated as a constant with a value \( r = 2.3 \) and similarly \( \Lambda = 0.8 \). Thus piezocone interpretations in clays can be linked with the state parameter approach for sands provided that the difference in drainage conditions between piezocone measurements in sand and clay are accounted for.

As noted in Chapter 4, approaches to account for drainage have been suggested by others. Houlsby (1988) pointed out the grouping \( Q (1-B_q)+1 \) was a simplification of the piezocone interpretation proposed by Konrad and Law (1987). It is argued that a normalized piezocone penetration resistance given by the grouping \( Q (1-B_q) \) exhibits relationships of the same form as equation 7.2
and, most importantly, the trend of coefficients $k$ and $m$ form an apparently single valued relationship with $\lambda$ regardless of piezocone drainage conditions when this grouping is used. The relationships between $k$, $m$ and $\lambda$ for the Author's data from Plewes et al. (1992) and the tailings database are shown in Figure 7.3 and can be approximated by the following equations:

$$Q_p = k \exp(-m\psi) = Q(1 - B_0)$$  \hspace{2cm} (7.4)

$$\frac{k}{M} = 3 + 0.85/\lambda$$  \hspace{2cm} (7.5)

$$m = 11.9 - 13.3\lambda$$  \hspace{2cm} (7.6)

where $M$ is the slope of the critical state line in terms of effective stresses and is equivalent to $[[\sin(\phi_c)][(3\sin(\phi_c))]^{-1}$.

### 7.3 Estimating In-Situ State of Mine Tailings

From examining equations 7.4 to 7.6, other than the piezocone parameters, the procedure only requires a value of $\lambda$ for every tailings evaluated by the piezocone. This is often impractical in many tailings (and natural soils) where gradation, and hence $\lambda$, can change significantly or where a great number of soil behaviour types are involved. Moreover, it is not always possible to carry out such extensive laboratory testing for time and/or economical reasons.
\[ Q(1 - B_q) = k \exp(-m \psi) \]

Figure 7.3 Relationship between Critical State Parameters and Material Compressibility
In general, tailings with a predominance of clayey minerals (e.g., tailings from micaceous-rich ores) exhibit large values of $\lambda$ compared to more particulate silts and sand tailings. At the same time, the friction ratio has been shown to vary according to not only gradation but type of fines (e.g. micaceous fines have higher $F$ than particulate fines). Therefore, it was considered not unreasonable to seek a direct relationship between the normalized friction ratio, $F$ and $\lambda$. Material index was not used but could likely equally have been related to $\lambda$. Figure 7.4 presents available information from the tailings database and several other sites involving silts and clays, as well as some laboratory chamber tests on sands. The other in-situ information and chamber test data is reproduced from Plewes et al. (1992). A first order fit to the data in Figure 7.4 is given by:

$$\lambda = F/10$$  \hspace{1cm} (7.7)

With an estimate of compressibility, equations 7.4 to 7.7, inclusive, provide the procedure for estimating the initial state of a given tailings where piezocone information is available. The method is readily applied as a screening technique to select samples for testing or to otherwise "target" weak layers. A value of $M = 1.2$ ($\phi_{cv} = 30^\circ$) may be reasonably assumed for most all tailings. This will fit most tailings typically within ±10%, although it may be slightly high for more plastic materials. Screening of tailings can be performed by using the equations to compute a value of $\psi$ directly as:

$$\psi = \frac{\ln \left[ \frac{Q(1-B_q)}{k} \right]}{-m}$$  \hspace{1cm} (7.8)
Figure 7.4 Proposed Relationship between F and Material Compressibility
or, if we take $M = 1.2$:

$$
\psi = \ell n \left( \frac{Q \left(1 - B_s\right)}{\left(3.6 + 10.2 / F\right)} \right) / \left(1.33F - 11.9\right) 
$$

(7.9)

Either of equations 7.8 or 7.9 can be readily included in automated interpretation software (e.g., is now included in the UBC-ISTG CPTINT v.5.0) or in less elegant post-processing work using, for example, spreadsheets.

Alternatively, the screening can be visually accomplished by plotting contours of $\psi$ on the piezocone soil behaviour type interpretation chart proposed in Chapter 6. These plotted contours are shown on Figure 7.5. This approach is preferred, and is in keeping with the use of the method as a screening procedure.

Figure 7.5 is significant in that it shows how in-situ state can be estimated for any combination of the three independent measurements from the piezocone. In Chapter 9, it is proposed that having such an estimate allows the screening-level prediction of key geotechnical parameters required in assessing mine tailings facilities.

### 7.4 Basic Validation of Methodology

One method for assessing the proposed procedure is to check the algorithm against existing experience in assessing piezocone data. Robertson (1990) made a significant step by suggesting that the piezocone interpretation plots carried useful quantitative data by showing a normally
Figure 7.5  Piezocone State Screening Approach
consolidated zone could be defined on a soil type classification chart. The procedure for estimating tailings state from piezocone data extends this concept from a different basis. The two approaches are compared using the approximate relationship in Equation 7.3 to transform \( \psi \) back to OCR with the condition of \( R=1 \) (e.g., normally consolidated). The results of this transformation are compared to the zone of normally consolidated behaviour indicated by Robertson (1990) and Jefferies and Davies (1991) on Figure 7.6. Good agreement between the two approaches is seen over the full range of soil behaviour types expected for tailings (e.g., sands, silty sands, silts and clays).

Another manner with which to assess the procedure can come from comparison with the empirical SPT database. Liquefaction susceptibility of cohesionless soils (e.g., most tailings) under seismic loading is most commonly made on the basis of stress normalized SPT blow count values \( (N_1)_{60} \) as proposed by Seed et al., (1987). Corrections for fines content are made to achieve an equivalent clean sand blow count value \( (N_1)_{60\text{-ECS}} \). Generally speaking, tailings deposits with \( (N_1)_{60\text{-ECS}} \) values of 30 or more are strongly dilatant and not normally considered susceptible to liquefaction. Tailings deposits with \( (N_1)_{60\text{-ECS}} \) values between 20 and 30 represent low liquefaction susceptibility under even the largest seismic events. Tailings with this in-situ density would experience only limited shear strains during the earthquake event and post-earthquake residual shear strengths are essentially peak values. Tailings with \( (N_1)_{60\text{-ECS}} \) values between 10 and 20 represent an intermediate susceptibility to liquefaction. Shear strains induced during liquefaction may be large in level ground conditions and residual shear strengths are often low enough to initiate slumping of embankment slopes. \( (N_1)_{60\text{-ECS}} \) values less than 10 represent a high liquefaction susceptibility; to both seismic and static transient loads. Very high shear strains can develop from small cuts in
Figure 7.6 Piezocone State for NC Soils - Check of Construct
otherwise level ground situations and flow slides can be produced in tailings impoundment from the very low residual shear strengths (see Chapter 9).

For comparing the state approach to the comprehensive SPT database, representative relationships between \(Q(1 - B_q)\) and \(F\) were developed for \((N_I)_{60-ECS}\) values of 10, 20 and 30. The relationships were calculated for a fine grained sand by making reasonable assumptions of \(F\), \(B_q\) and \(q_v/N_60\) (see Chapter 8) and \(\Delta N_{corr}\) (Seed and Harder, 1990) for various fines contents. The method, assumptions and calculated values of \(Q(1 - B_q)\) and \(F\) are provided in Table 7.1.

### Table 7.1 Calculation of \(Q_p\) and \(F\) for Representative Values of \((N_I)_{60-ECS}\)

<table>
<thead>
<tr>
<th>Fines Content (%)</th>
<th>(F) (%)</th>
<th>(q_v/N_60)</th>
<th>((N_I)_{60-ECS} = 10)</th>
<th>((N_I)_{60-ECS} = 20)</th>
<th>((N_I)_{60-ECS} = 30)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(N_60)</td>
<td>(q_v) (bar)</td>
<td>(Q(1 - B_q))</td>
<td>(N_60)</td>
<td>(q_v) (bar)</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>4.5</td>
<td>10</td>
<td>45</td>
<td>43</td>
</tr>
<tr>
<td>15</td>
<td>0.6</td>
<td>4</td>
<td>6.5</td>
<td>26</td>
<td>24</td>
</tr>
<tr>
<td>35</td>
<td>1.0</td>
<td>3.5</td>
<td>3</td>
<td>11</td>
<td>9</td>
</tr>
</tbody>
</table>

Notes: 1. \(B_q\) @ 0 assumed for all cases.
2. Vertical effective stress, \(\sigma_v'\) = 1 bar.
3. Total vertical stress, \(\sigma_v = 2\sigma_v'\ = 2\) bar.
4. \(N_60 = (N_I)_{60} \sqrt{\gamma_v}\)
5. \((N_I)_{60} = (N_I)_{60-ECS} - \Delta N_{corr}\)

Figure 7.7 plots the position of the lines representing \((N_I)_{60-ECS}\) values of 10, 20 and 30 on the proposed screening chart. The relationships shown on Figure 7.7 represent the conditions assumed in Table 7.1 only. However, the positions of the lines are not very sensitive to minor changes in the assumed parameters.
Figure 7.7  Proposed Piezocone State Screening Compared to SPT \((N_1)_{60\text{-ECS}}\)
Good correlation is observed between the predicted liquefaction hazard from the SPT based method and the piezocone state based screening method. Conventionally, strong dilatant behaviour would be predicted for \( \psi \) values < -0.1 which reasonably corresponds to \((N_l)_{60-ECS} > 20\) in Figure 7.7, representing low to negligible liquefaction susceptibility. Weak, potentially metastable states would exist for \( \psi > 0 \) and tailings in this range would be predicted to be most susceptible to liquefaction. This is in very good agreement with case histories of soils as represented by \((N_l)_{60-ECS} < 10\) from SPT methods in Figure 7.7.

Very approximately, at least to values of \( I_c < 2.5 \), values of \( \Psi \) can be given an SPT \((N_l)_{60-ECS}\) equivalent as shown in Table 7.2. Table 7.2 is developed from the definition of \( \Psi \) given in this chapter and the piezocone estimated SPT values as described in Chapter 8.

<table>
<thead>
<tr>
<th>( \Psi )</th>
<th>((N_l)_{60-ECS})</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.2</td>
<td>&gt;35</td>
</tr>
<tr>
<td>-0.1</td>
<td>20 to 30</td>
</tr>
<tr>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>+0.1</td>
<td>&lt;2</td>
</tr>
</tbody>
</table>

### 7.5 In-Situ Assessment of Methodology

One of the suggested key uses of a state parameter derived from piezocone data is for use in liquefaction susceptibility screening of tailings deposits. The general procedure is illustrated by two case histories involving hydraulically deposited mine tailings. Both case histories consist of static liquefaction of loosely deposited sand during tailings dam construction. The liquefaction events
occurred at the Sullivan Mine (see Chapter 9 and Davies et al., 1998) and at the Suncor tailings facility in Alberta (Plewes et al., 1988).

At Suncor, an upstream static liquefaction event occurred over a 300-m wide section of a tailings dam and involved tailings deposited in a loose state below water. \((N_i)_{60-ECS}\) values of 8 to 10 were measured after the event in the loose tailings adjacent to the failure zone. Piezocone sounding data adjacent to the failed section are presented in the unified state form in Figure 7.8. The piezocone data in Figure 7.8 indicates that a significant fraction of data falls below the \(\psi = 0\) line indicating that the tailings were in an initially loose, metastable state. Hence, cause for reasonable concern over the potential for liquefaction of the tailings was accurately predicted by the screening procedure. The \((N_i)_{60-ECS}\) of the tailings is predicted from Figure 7.7 to be in the order of 7 to 10 and directly agrees with the field measured values.

At the Sullivan Mine, a relatively large downstream static liquefaction event occurred in the silt tailings (50% to 60% passing No. 200 sieve) of the Active Iron Pond (AIP). More details on this event are presented in Chapter 9 and Davies et al. (1998). The piezocone soundings adjacent to the event, shown in Figure 7.9, showed that a pervasive layer in the foundation tailings was in an extremely loose condition. Raw \((N_i)_{60}\) values were in the order of 0 to 2, with \((N_i)_{60-ECS}\) of 6 to 10 (Davies et al, 1998). Again, the \((N_i)_{60-ECS}\) values are in good agreement with Figure 7.7.

As further in-situ assessment of the methodology, the screening procedure was applied to a series of piezocone soundings obtained along the length of a hydraulically deposited tailings beach (Syncrude Mildred Lake toe berm). This example is used to demonstrate the ability of the
technique to differentiate between soils deposited in states ranging from loose to moderately dense. Figure 7.10 shows the range of piezocone data obtained in the upper beach zone near the point of tailings discharge, middle beach zone and the lower beach near the water reclaim pond. The lower beach data also includes some tailings deposited below water.

As noted in Chapter 3, the hydraulic deposition of tailings creates a situation whereby the beach becomes finer in composition with distance from the point of tailings discharge. This known phenomena is readily observed in the classified piezocone data as the tailings vary in gradation from a clean to silty sand near the discharge point to a predominantly silt material near the reclaim pond (e.g., $I_c$ values from ~1.25 to ~3.0). The sand deposited near the discharge point represents competent material with $\psi$ values of less than -0.1 and $(N_d)_{60-ECS}$ of 20 or more. The high density is attributed to the low percentage fines (10% to 20% passing No. 200 sieve) and high energy mode of deposition. The middle beach tailings incorporate a slightly higher overall percentage of fines and are deposited in a lower energy mode. As a consequence, higher $\psi$ values are associated with the tailings in the middle beach region. The tailings deposited near the reclaim pond consists of silty sand deposited on the margins of the reclaim pond, and fine silt and clay sedimented from standing water within the pond. The deposition of these materials occurs under quiescent conditions resulting in low relative density of the cohesionless deposits and very soft consistency of the cohesive deposits. These materials represent stability concerns under static and dynamic loading. This is corroborated by the high $\psi$ values in Figure 7.10.
Figure 7.8  State Screening for Suncor Tailings Piezocone Data near Liquefaction Slump
Figure 7.9  
State Screening for Loose Zone of Sullivan Tailings near Liquefaction Slump
Figure 7.10  Piezocone Screening Applied to Hydraulic Tailings Beach Piezocone Data
7.6 Tailings Database Validation of the Methodology

The state approach was applied to all of the piezocone soundings in the database. Where a particular minesite yielded data from several locations relative to the tailings discharge location, similar distributions to that shown in Figure 7.10 were evident. Further application of state approach to the tailings database is discussed in Chapter 9 where the approach is suggested as a method for screening liquefaction susceptibility of mine tailings.

As an additional validation of the proposed state approach, the CANLEX project utilized the Author's algorithm to assess in-situ density. The standards for comparative purposes were in-situ frozen samples, with some density adjustment for freeze-thaw effects, and nuclear logging methods. For the most part, the approach was shown to work quite well. For example, Figure 7.11 shows a summary of the predicted state parameter versus the value in-situ as assessed by the frozen sampling program. There appears to be quite good agreement between measured and predicted values with an error to about +/- 0.03 units in void ratio space. This level of error should be appreciated when developing screening level assessments using the state parameter methodology described in this thesis.

7.7 Additional Comments

The piezocone interpretation framework provided in Chapter 4, and then augmented with the concepts of material index and compressibility-adjusted state estimate in Chapters 6 and 7, is close to a unified interpretation basis for all soils of which mine tailings are a special subset. Been and Jefferies (1993) provided an initial attempt of linking material compressibility with a material index (and hence state). Figure 7.12 is adapted from that reference but shows a
Figure 7.11 Estimated State Parameter from Proposed Piezocone Approach and CANLEX Frozen Samples (adapted from Robertson and Wride, 1997)
Figure 7.12 Unification of State Concept

LEGEND
- LK BONNEVILLE SILT*
* ST CLAIR TILL*
+ Tarsiut P45, Units A, B1, B2, B3*
□ Amauligak Pipeline Route*
▲ Yatesville Sand (Brandon et al, 1990)
× Clean Sands (Erksak, Ticino and Syncrude)
● Alaskan Tailings
▼ Kuc Tailings (Shell)
⊙ Endako Tailings (Dyke)
■ Highland Valley Copper
△ Sullivan Iron Tailings (Approximate)

* BEEN AND JEFFERIES (1993)
suggested expansion to the concept. The line B shown in Figure 7.12 was that proposed by Been and Jefferies (1993) based upon their limited database. Although the tailings database does not substantially increase the available data, the contributions do suggest that a single relationship may be too generic. For example, the carbonate-rich Alaskan tailings, and many other angular tailings, have more compressible skeletal matrices when compared to standard alluvial soils. Consequently, it should not be surprising to see a material index versus compressibility relationship place these and other angular tailings below the “line” for more traditional fluvial or hydraulic fills (more compressible per given $Q(1-B_q)$ - F pairing). At the same time, some materials such as sub-gravel glacial tills could be expected to exhibit the opposite trend; i.e. being more incompressible for given material index.

Figure 7.12 includes an extension of the concept by Been and Jefferies (1993) to include the limited, but observable trends, in the database. From equation 7.7, which is a very simple first-order empirical compressibility correlation, a relationship between material index and compressibility should be expected. As an initial attempt to establish a relationship between material index and material compressibility, the following is suggested per Figure 7.12:

$$1/\lambda = \Delta - 10 (I_c)$$

where: $\Delta = 41$ for low compressibility soils (e.g. sub-gravel till)
34 for normally compressible soils (e.g. most fluvial sediments and some mine tailings)
28 for highly compressible soils (e.g. angular mine tailings)
The database relating material index with material compressibility needs considerable augmentation. For mine tailings specifically, it is suggested that a relationship in the form of equation 7.10 should replace equation 7.7, with the appropriate value of $\Delta$, as the means of estimating $\lambda$ in the computation of in-situ state, $\psi$.

Robertson and Fear (1995) suggested a relationship in the form shown in Figure 7.13 to estimate material compressibility from seismic piezocone data. This relationship is developed between compressibility (natural log) and an empirical value, $A$, defined as:

\[
q_{c1} = \frac{(V_{s1})}{A}^4
\]  

(7.11)

where: 
- $q_{c1} = q_c (Pa/\sigma_{vo})^{0.5}$
- $Pa$ = atmospheric reference pressure
- $q_c$ is in Mpa
- $V_{s1}$ is in m/s

The parameter $A$ is noted as being controlled by grain characteristics, matrix compressibility, age and degree of cementation. For mine tailings, age is not typically a consideration. However, chemical cementation can be a very important component of material response as will be noted in Chapter 9 regarding the Sullivan Mine tailings shear wave velocity response.

The tailings database could not be used to adequately assess the proposed relationship in Figure 7.13. Available data from tailings show that for uncemented tailings, e.g. at Endako, the relationship could be argued to be acceptable. However, for the KUC tailings, where chemical cementation from oxidation processes has stiffened the matrix to very low shear strains, the relationship appears invalid. Approximate data from the Sullivan tailings
$q_{c1} = \left(\frac{V_{s1}}{A}\right)^4$

where:
- $q_{c1}$ (MPa)
- $V_{s1}$ (m/s)

Figure 7.13 Material Compressibility from Seismic Piezocone (adapted from Robertson and Fear, 1995)
and Highland Valley Copper Tailings also shows some lack of agreement with the fourth-order relationship in Figure 7.13. Although the conclusion should be viewed as preliminary, the approach suggested in Figure 7.13 should be used with caution for mine tailings.
8. STATIC LOAD GEOTECHNICAL PARAMETERS

8.1 Overview

The piezocone can provide estimates of a number of geotechnical parameters for static loading conditions including drained strength of cohesionless soils and undrained strength of cohesive soils. This research does not extend the existing relationships for parameter estimation. However a method to predict SPT blowcounts directly from piezocone data without the need for a physical sample is presented. The ability to estimate SPT values with a good degree of confidence allows access to the considerable SPT database of engineering parameters. The piezocone SPT algorithm presented was originally developed by the Author at the outset of the research (Jefferies and Davies, 1993) and evaluated to ensure an appropriate fit to the tailings database.

8.2 Piezocone for Estimating Parameters

One of the primary uses of the piezocone in standard geotechnical practice has been to provide estimates of soil parameters for design purposes. For static loading geotechnical issues, this mainly involves providing estimates of drained or undrained parameters for both strength and modulus character.

For evaluating conventional soils with piezocone data, Lunne et al. (1997) and Campanella and Robertson (1988) are among several good basic references for the geotechnical engineer. However, specific relationships for mine tailings are not readily available and it is uncertain whether global soil relationships are valid for mine tailings.
8.3 Correlation between the Piezocone and Standard Penetration Test

8.3.1 Proposed Methodology

The geotechnical database for static load parameters from piezocone data is still largely influenced by the standard penetration test (SPT) database. In many cases, the relationships developed for piezocone data have been an extension of an existing SPT relationship by means of a relationship between piezocone data and the SPT data. Traditional methods of relating piezocone data to SPT data have solely used tip resistance and non-normalized parameters. An alternative approach is suggested.

Penetration tests are commonly used to test soils, and there is considerable knowledge relating soil behaviour to the SPT (ASTM Method for Penetration Test and Split-Barrel Sampling of Soils, D 1586). However, the piezocone offers a test with many advantages over the SPT including: precision, repeatability, continuous logging, multiple channel measurements, and ease of use in offshore testing. However, as noted, it is often desirable to refer to the SPT-based experience record when using the piezocone. To achieve this, a mapping between the two types of penetration tests is required.

The relationship between the piezocone, represented by the tip resistance \( q_c \) and the SPT, represented by the blowcount \( N \), has been determined in a number of studies over the past 30 years (e.g., Meigh and Nixon, 1961; Thornburn, 1970; Schmertmann, 1970; Burbidge, 1982; Robertson et al. 1982; Seed and de Alba, 1984; Burland and Burbidge 1985). The relationship between the piezocone and SPT is best expressed in terms of the ratio \( q_c/N \) (MPa/blows per 300 mm). CPTINT presents a table of \( q_c/N \) values related to the soil behaviour type zones of...
Robetson et al. (1986). The $q_c/N$ data from available literature is summarized in Figure 8.1 in terms of the average particle size of the soils tests. Plotting data in this manner assumes the relationship between the two tests is functionally dependent only upon soil type as characterized by average particle size, a tacit assumption underlying previous studies. The database is also hampered by the lack of adherence to SPT energy calibration requirements which results in a much wider scatter in literature data than is necessary.

Although Figure 8.1 appears to be suitable to estimate SPT results from data, this is only the case if sampled boreholes are available with grain-size data in the various strata. This also relies on extreme lateral homogeneity of deposits, a condition which is not common in tailings or many natural materials. What is really required is a relationship between SPT and the piezocone based on piezocone data alone; such a relationship would permit estimation of SPT resistance without boreholes or samples. A direct relationship based on piezocone parameters alone would also avoid the uncertainty introduced by soil gradation changing between the piezocone data and the supposed corresponding soil sample.

As noted in Chapter 6, mine tailings specifically and all soils in general, can have their relative soil behaviour type denoted by their material index, $I_c$, given as:

$$I_c = \sqrt{\left\{3 - \log Q(1 - B_q)\right\}^2 + \left[1.5 + 1.3 (\log F)\right]^2}$$ \hspace{1cm} (8.1)
Figure 8.1 Summary of Traditional SPT - Piezocone Database (adapted from Burland and Burbidge, 1985)
Based upon the data from Figure 8.1 and Table 6.1, and mine tailings database, a relation between $q_c/N$ and $I_c$ can be established assuming consistency in the soil behaviour types established in Chapter 6. Figure 8.2 presents this relationship where material index is used in place of soil gradation. Also plotted on Figure 8.2 is the inferred range of data from the Robertson (1990) soil behaviour type classification. The fact that the relationship was found to be somewhat higher than most published relationships is not surprising; a bias towards finer-grained materials is common (Jefferies and Davies, 1993). The resulting relationship from Figure 8.2 is given as:

$$\frac{q_c (MPa)}{N \text{ (blows per 300 mm)}} = 0.85 \left(1 - \frac{I_c}{4.75}\right)$$  \hspace{1cm} (8.2)

Consequently, equations 8.1 and 8.2 form the proposed algorithm for estimating SPT data from the piezocone. The parameter values used as characteristic of the piezocone are the measured values averaged over the 300 mm interval corresponding to the SPT.

A further advantage of using the proposed algorithm over previous efforts exists in the computational simplicity. Equations 8.1 and 8.2 can be combined into a single expression to yield $N_{60}$ values from piezocone data during automated evaluations. The combined expression can be placed in existing piezocone analyses codes or, perhaps less elegantly, within a spreadsheet. Earlier methods do not offer this ease of automation feature.
Figure 8.2  Relationship between Piezocone Material Index and $q_c/N_{60}$ (adapted from Jefferies and Davies, 1993)
Further details on the formulation of the initial algorithm and its replication trials (including checks for systematic bias toward Q, F and depth) can be found in Jefferies and Davies (1993).

8.3.2 Tailings Database Evaluation of Proposed Methodology

Following the initial algorithm development, several of the research locations had high quality SPT data for specifically calibrating the relationship for mine tailings. At Syncrude’s Mildred Lake Settling Basin (MLSB), energy calibrated SPT results were obtained by the Author as part of the CANLEX project. The Author in UBC-ISTG (1994d) provides details of the investigations.

A summary of the details of the SPT program is as follows. The boreholes were advanced by rotary drilling rig using a 11 cm tricone bit and AW drill rods. The drill rods were assumed to have a unit mass of 2.18 kg/m. Casing was required in the upper 1.5 metres to prevent caving of loose surface material. Mud circulation was used to maintain borehole integrity. Two different safety hammers were used in the program. Both of these were weighed and found to adhere to the established 63.5 kg (140 lb.) standard weight. The hammers were operated via a rope and cathead system; the rope was relatively new hemp rope and was not wetted during the testing. The SPT-N values were determined by summing the number of blows to penetrate the final 300 mm of a 450 mm penetration depth. Soil liners were not used.

Other relevant details specific to the boreholes and the SPT tests can be found on the field logs for CNLX 9406 and CNLX 9407 included in UBC-ISTG (1994d).
To carry out the energy calibration, an instrumented rod of 0.5m length was used at the top of the drill-rod string to measure the force and velocity imparted by a blow. The instrumentation included two load cells and an accelerometer. The load cells were used to record the time histories of the force delivered blow and the accelerometer signals were integrated to obtain the corresponding velocity-time histories during each hammer blow. The high speed data recording system allowed the energy to be measured for every blow up to a maximum of 41 blows for each test depth.

The through-put energy actually being transferred to the drill-rods was determined using the "force-squared" ($F^2$) and the "force-velocity" (FV) methods. Complete details for each method can be found in ASTM D 4633-86 and in Sy and Campanella (1991), respectively.

The results from both analytical methods, $F^2$ and FV, indicated that the energy delivered varied around an average value of between roughly 65% and 70% for both boreholes. This average value is typical for safety hammers and is usually higher than the value of 55% used for donut hammers. Overall, relatively constant energies were experienced over two different days with two different hammers.

Figure 8.3 presents a comparison between the measured results and the average $N_{60}$ values estimated from the adjacent piezocone data. The piezocone estimated SPT blowcounts are very close to those actually obtained under the research conditions.
Figure 8.3  Measured versus Predicted $N_{60}$ Values - SCL MLSB
Similar good correlations were found with the remainder of the tailings database. As such, the algorithm per Equation 8.2 appears valid for at least the range of tailings assessed in this research. Figure 8.4, for example, shows a comparison between the piezocone estimated $N_{60}$ values at the crest area of Pond No.1 (North Dam) at the Endako Minesite and an energy calibrated SPT program carried out by others. The drilling was carried out using a Mayhew 1000 Drilling Rig, 12 cm casing and standard SPT equipment without liners. Energy calibration was used per the $FV^2$ method which yielded energies between 64% and 84% of the theoretical maximum. Once corrected to $N_{60}$ values, Figure 8.4 shows the good agreement between the information from BH96-2 and piezocone sounding ENDK9601 located within five metres of BH96-2.

The methodology was also developed/calibrated on some of the more “challenging” materials in the database such as the finer-grained, cemented (brittle) tailings present at the Sullivan Mine. For example, in an area of the Old Iron Pond close to the backscarp of the 1948 static liquefaction slump, energy calibrated SPT data was available. This data is compared to the algorithm as shown in Figure 8.5. Again, the estimated $N_{60}$ values are considered quite acceptable.
Figure 8.4  Measured versus Predicted $N_{60}$ Values - Endako Pond No. 1
Figure 8.5  Measured versus Predicted $N_{60}$ Values Sullivan Active Iron Pond
8.4 Static Drained Parameters

For the drained parameters of mine tailings from piezocone data, no specific correlations were developed. Those typically applicable to sands and silty sands (e.g. using CPTINT) appeared to provide reasonable estimates of the parameters for any initial design requirements.

Table 8.1 provides summarized results from a variety of CPTINT interpretations of different tailings from the research database. The parameters presented are those typically required by engineers for routine calculations of static behaviour; drained strength, a moderate strain secant modulus of elasticity and approximate relative density (state). Clearly, using the seismic piezocone coupled with more definitive laboratory or specialized field testing is preferable for accurate modulus computations. Moreover, as shown elsewhere in this thesis, state parameter approaches are more relevant than estimates of relative density such as shown in Table 8.1.
Table 8.1 Estimated Static Geotechnical Parameters for Tailings Using Established Relationships for General Soils

<table>
<thead>
<tr>
<th>Tailings Material</th>
<th>Estimated $e_{@10 \text{ m depth}}$</th>
<th>Estimated $E_{25, \text{ @10 m depth}}$</th>
<th>Estimated $D_r, % \text{ @10 m depth}$</th>
<th>Estimated $S_u/S_o$, N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Endako Pond 1 Dam</td>
<td>37</td>
<td>210</td>
<td>55</td>
<td>N/A</td>
</tr>
<tr>
<td>Endako Pond 2 Dam</td>
<td>39</td>
<td>240</td>
<td>60</td>
<td>N/A</td>
</tr>
<tr>
<td>Sullivan AIP</td>
<td>30</td>
<td>80</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>Sullivan Siliceous</td>
<td>37</td>
<td>100</td>
<td>25</td>
<td>0.6</td>
</tr>
<tr>
<td>Sullivan Gypsum</td>
<td>43</td>
<td>140-280</td>
<td>65</td>
<td>N/A</td>
</tr>
<tr>
<td>Sullivan Calcine</td>
<td>31</td>
<td>50-75</td>
<td>20-30</td>
<td>N/A</td>
</tr>
<tr>
<td>Sullivan Old Iron Pond</td>
<td>37</td>
<td>160-200</td>
<td>45</td>
<td>N/A</td>
</tr>
<tr>
<td>Trail Sand and Gravel</td>
<td>45-47</td>
<td>550-700</td>
<td>110</td>
<td>N/A</td>
</tr>
<tr>
<td>Gibraltar Dam</td>
<td>42</td>
<td>340-440</td>
<td>90</td>
<td>N/A</td>
</tr>
<tr>
<td>Gibraltar Beach</td>
<td>29</td>
<td>65-90</td>
<td>30</td>
<td>N/A</td>
</tr>
<tr>
<td>INCO Pistol Dam</td>
<td>36</td>
<td>110</td>
<td>65</td>
<td>N/A</td>
</tr>
<tr>
<td>INCO Pistol Beach</td>
<td>30</td>
<td>75-100</td>
<td>35</td>
<td>0.35</td>
</tr>
<tr>
<td>INCO Checetto Dam</td>
<td>37</td>
<td>125</td>
<td>70</td>
<td>N/A</td>
</tr>
<tr>
<td>INCO Checetto Beach</td>
<td>32</td>
<td>60-100</td>
<td>30</td>
<td>0.35</td>
</tr>
<tr>
<td>SCL Mildred Lake (Cell 24)</td>
<td>36</td>
<td>180</td>
<td>55</td>
<td>N/A</td>
</tr>
<tr>
<td>SCL SWSS BAW</td>
<td>39</td>
<td>360-420</td>
<td>70</td>
<td>N/A</td>
</tr>
<tr>
<td>SCL SWSS BBW</td>
<td>33</td>
<td>80-200</td>
<td>40</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Notes: 1. Using Campanella and Robertson (1983)
2. Using Campanella and Robertson (1983)
4. $N_{KT}$ method where $N_{KT} = 10$. For tailings with $I_c > 3$ and measured, or suspected, PI > 10.

The values in Table 8.1 should be viewed as augmenting similar literature for obtaining initial estimates of geotechnical parameters for design overview.

8.5 Static Undrained Parameters

As with drained parameters, there was no specific attempt to adjust the undrained parameter estimates from the global values present in the more commonly cited references. At the same time, where undrained conditions were noted by the piezocone interpretation, undrained parameters were available through such global estimates. Little site specific correlative data was available to compare values with these estimates although there was some undrained strength
information available (largely from in-situ vane tests) on the more cohesive fine-grained tailings in the database. Although certainly not statistically significant, from the available data, the $N_{KT}$ approach noted below using a value of $N_{KT} \approx 10$ in Equation 8.3 - appeared to provide the most consistently sensible results.

$$S_u = \frac{(q_t - \sigma_{vo})}{N_{KT}}$$

(8.3)

### 8.6 Summary

Other than the proposed SPT estimation algorithm, which is based upon the normalized piezocone parameters suggested in Chapter 4, static load geotechnical parameters were not a prime focus of this research. Mine tailings structures, from a geomechanical perspective, have seldom demonstrated internal (non-foundation related) static load concerns if we do not include "static" liquefaction in the consideration. For the most part, it is transient loading conditions (including those which bring about "static" liquefaction) which are of a much greater importance to the designers and operators of mine tailings facilities. The concepts of in-situ state and the corresponding minimum assured strength upon shear loading are the essential ingredients for adequate design and stewardship of these facilities. Therefore, the static load parameters are simply an extension of the concepts developed in Chapters 6 and 7 with the strength concerns addressed per the transient loading considerations in Chapter 9.
9. TRANSIENT LOAD GEOTECHNICAL PARAMETERS

9.1 Overview

This chapter presents two main original contributions:

- A liquefaction susceptibility screening method for tailings based on the state parameter; and
- A method to predict the post-liquefaction strength of tailings.

In addition, a case history is introduced, the 1991 Sullivan Mine static liquefaction failure, which assists in the assessment of the two contributions.

The geotechnical literature pertaining to the liquefaction phenomena is immense. This chapter does not attempt to capture the essence of that body of work but instead focuses on the more narrow evaluation of the two contributions noted above. There is no implication that more traditional field-based liquefaction assessment methods are inferior; it is simply that they were not addressed by this research.

The aim of the research was to identify for procedures to assess liquefaction susceptibility of tailings, particularly for non-seismically induced liquefaction, and assign a post-liquefied strength. The prerequisite of such procedures was that they utilized piezocone data alone and could be favourably assessed against actual tailings field performance.
9.2 Definitions

Transient loads pose the greatest geomechanical concern to mine tailings facilities and these facilities are subject to a number of transient loading conditions. The most readily identified of these conditions, at least from a design perspective, is the transient loading from seismic events. Whether limited-deformation or eventual flowslide development, the effects of transient seismic loads on mine tailings are well-recognized by current engineering standards. However, there are other transient loads that affect mine tailings which can be of equal importance to seismic loads due to their common occurrence at mine sites in comparison to seismic events. Included in these common transient loads are incremental impoundment raise construction and episodic tailings slurry placement. The former can lead to relatively rapid increases in stress levels and undrained conditions in susceptible materials while the latter can cause temporary changes to the amount of tailings saturated in a given section of an impoundment. Conversely, static loads are taken to be those in place for a considerable period.

Regardless of loading condition, the most dramatic effect a transient load can have on mine tailings is to impart liquefaction of those tailings over a volume that leads to a “failure” event. “Failure” can mean different things but non-intentional release of tailings solids or supernatant fluid(s) is the most dramatic failure mechanism and the one most typically set as the design “upset” condition. The term transient load is chosen to avoid the confusion between seismic and static liquefaction events because, though the loading conditions are different, the resulting concern to the mine operator is identical.
9.3 Small Strain Dynamic Modulus and Damping

In developing a constitutive understanding of any geological material, the small strain elastic response is of fundamental importance as it is the "backbone" to the relationship. However, as noted elsewhere in this thesis, knowledge of this small strain modulus does not provide any particular indication of a soil's large strain behaviour. Other information, which can be obtained by in-situ strain/stress controlled testing (e.g. self-boring pressuremeter) or laboratory testing on appropriate samples, is required to develop the necessary parameters for appropriately predicting the character of a soil at larger (i.e. non-elastic) strains.

During the tailings research, the seismic piezocone was used at every minesite with a total of about 20 profiles being developed. As described in Chapter 4, the shear wave velocity, $V_s$, was calculated for these profiles by dividing the difference in travel distance between two depths by the time difference between the two-recorded signals. Time difference was determined in two fashions. First, it was found manually by picking either the arrival time of the main shear wave pulse or by the crossover technique. Alternatively, the time lag can be taken as the time shift of the maximum cross-correlation of the signals. The time can also be calculated as a function of frequency, $f$, using the phase of the cross spectrum of the signals. Dividing this time into the difference in travel distance gives the velocity as a function of frequency, $V_s(f)$. To do this latter computation, detailed signal processing is usually required.
Figure 9.1 shows a composite of the shear wave velocity profiles from several tailings. Although there is some scatter, particularly with the Sullivan Mine data, the trend in the data is quite noticeable. Overall, computation of $G_{\text{max}}$ from such data would not be expected to produce dramatically different results between the materials save, again, for the oxidized Sullivan Iron tailings. The agreement in trend is even more dramatic in Figure 9.2 which presents stress normalized (per Robertson and Wride, 1997) shear wave velocities. The seemingly anomalously high values for the Sullivan data was actually found quite commonly in the sulphide mine tailings tested during the research. As will be discussed later in this Chapter, oxidation processes in these tailings likely produce matrix stiffness from chemical bonding. As discussed later, the presence of these bonds tend to invalidate attempts to use $V_s$ as an indicator of large strain behaviour.

Beyond the measurement of shear wave velocity and modulus, the seismic piezocone can also be used to further evaluate the dynamic response of the ground. In general, the intensity of a seismic wave decreases as distance from its initial source increases due to wave attenuation. Wave attenuation is mainly due to geometric spreading and energy dissipation within the soil mass caused by material damping. Attenuation is most commonly modeled as viscous damping, where the simulated damping force is assumed to be proportional to the velocity within the given soil element. The resulting constant of proportionality is termed the coefficient of viscous damping. The damping ratio, $D_s$, is defined as the ratio of the coefficient of viscous damping to a critical value of the coefficient where the motion is attenuated within one-half of a wave cycle. The subscript, $s$, is used to indicate that the parameter is used for characterizing the behavior of
Figure 9.2  Stress Normalized Shear Wave Velocities - Several Mine Tailings
the shear waves. The damping ratio is an important soil property required for dynamic analyses when simulating the unload-reload behavior of a soil column subjected to transient loading conditions. Having a site-specific value of the damping ratio versus an assumed value can significantly improve the relevance of dynamic analyses.

For routine shear wave velocity measurements, detailed signal processing is not usually required and the velocities can be easily determined from arrival times in most cases. However, for damping measurements, it is necessary to evaluate the attenuation of amplitude of the signal with time. Such a signal processing also leads to an understanding of the properties of the measured signals.

Briefly, the signals obtained from the seismic piezocone were systematically processed by:

- reviewing the average of several tests (typically hits with a set-distance drop hammer on the ISTG vehicle);
- removing noise on sites where signal contamination can be a problem with a low pass filter with a flat response up to about 100 Hz being found to be sufficient in most cases;
- selecting the main shear wave pulse from the signal;
- windowing the signal so that the selected waveform contains all the characteristics of only the shear wave (a tacit assumption); and
- reviewing the Fourier transforms for each layer identified.

Following these steps, the windowed main shear wave pulse was used as the input signal for computing $V_s(f)$. Once the $V_s(f)$ data is available, determining the small strain damping ratio is
relatively straightforward. The basis of the methodology is the Spectral Ratio Slope (SRS) which has been previously developed and used by others for interpreting seismic waves (Campanella and Stewart, 1992).

The damping ratio can be evaluated using Equations 9.1 and 9.2 per:

\[
k = \frac{\partial^2 \left( -\ln \frac{A_R}{A_0} \right)}{\partial f \partial d}
\]

(9.1)

and

\[
D_s = \frac{k V_s}{2 \pi}
\]

(9.2)

where

- \( f \) = frequency, Hz,
- \( d \) = depth, m
- \( \partial/(\partial d) \) = slope with respect to depth,
- \( \partial/(\partial f) \) = slope with respect to frequency,
- \( A_0, A_R \) = amplitudes of the Fourier transforms of a reference signal and the signal at a depth where the damping ratio is to be evaluated,
- \( V_s \) = the shear wave velocity of the layer,
- \( A_R/A_0 \) = Spectral Ratio,
- \( D_s \) = the small strain damping ratio of the layer (decimal).

The parameter \( k \) has units of \( \text{(time/length)} \). The derivation of these equations can be found in Campanella and Stewart (1992). The essential feature of the double differentiation (first with respect to frequency and then with respect to depth) is that the radiation damping component is eliminated from the expression. Thus, it is not necessary to assume or attempt to evaluate the amount of radiation damping at each depth. However, a variation of this approach can be used to study the distribution of radiation damping with depth in different soils. The procedure for
implementing these expressions to evaluate the small strain damping ratio is given by Campanella and Davies (1994) with Campanella et al. (1994) providing worked examples.

Although not computed for every site within the research database, $D_s$ was computed for several of the tailings. The available literature for equivalent values is essentially non-existent so comparisons with natural mineral soils tested by the ISTG are included in Table 9.1 which presents a summary of typical results from the research.

### Table 9.1 Typical Values of Low Strain Damping Ratio for Standard Soils and from Tailings Database and Related ISTG Information

<table>
<thead>
<tr>
<th>Material</th>
<th>Damping Ratio ($D_s$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ottawa Sand</td>
<td>0.5%</td>
</tr>
<tr>
<td>Silty Clay</td>
<td>2.0%</td>
</tr>
<tr>
<td>Syncrude Sand Tailings</td>
<td>0.7%</td>
</tr>
<tr>
<td>Sullivan Iron Tailings</td>
<td>1.2%</td>
</tr>
<tr>
<td>Endako Molybdenum Tailings</td>
<td>0.7%</td>
</tr>
<tr>
<td>Lower Mainland B.C. Peat*</td>
<td>2.0%</td>
</tr>
<tr>
<td>Lower Mainland B.C. Organic Clay*</td>
<td>3.0%</td>
</tr>
<tr>
<td>Lower Mainland B.C. Silty Clay*</td>
<td>1.0%</td>
</tr>
</tbody>
</table>

* Lower Mainland values from Campanella et al. (1994)

The values determined for all the tailings materials tested showed intermediate behaviour between a sandy soil and a clayey soil. This could be argued as not being unexpected given the gradation and mineralogy of the tailings involved. At the same time, the Sullivan Mine Iron Pond tailings showed a higher damping ratio than might have been predicted. Overall, the number of data points available was small and the results are, at best, preliminary. Consequently, the values in Table 9.1 can be used for preliminary assessments of tailings facilities where
tailings similar to those in the tailings database exist. However, site specific determination of $D_s$ is recommended where seismic piezocone data is available.

9.4 The Liquefaction Phenomena

9.4.1 General

Over the past two decades, issues related to liquefaction have become one of the more heavily researched and published sub-disciplines of soil mechanics. Liquefaction flow failures of mine tailings represent some of the more dramatic case history contributions to this list of publications. The definition developed by the NRC (1985) for liquefaction and its related physical phenomenon is used in this thesis.

No attempt to duplicate the extensive literature on liquefaction will be attempted herein. Tailings will be assumed to have one of three characteristics upon shear loading:

1. strain softening (full liquefaction with the potential for limitless deformation);
2. limited strain softening (limited liquefaction with limited deformation); and
3. strain hardening (no appreciable liquefaction or deformation).

The liquefaction equivalence noted above for each loading condition is consistent with the NRC (1985) nomenclature.
Figure 9.3 presents schematic representations of these three possible responses to monotonic shear loading conditions. Identical responses, although with more complex loading histories, can result from cyclic shear loading. Figure 9.3 includes the concept of collapse surface used in its most general sense. The Author acknowledges the debate regarding the position, linearity and even existence of the collapse surface. Nonetheless, in compressive triaxial loading of loose tailings samples with grain imbrication normal, to subnormal, to the principal effective stress, such a “surface” can be replicated during laboratory testing programs. The inclusion of the concept of a collapse surface in Figure 9.3 is for illustrative purposes only and offers a simplistic framework for the discussions which follow.

9.4.2 Liquefaction and Mine Tailings

Tailings facilities, in every major jurisdiction in the world, are subject to specific guidelines with respect to addressing the potential for seismic liquefaction. For seismic liquefaction to occur, the number of significant cycles which are imparted by the earthquake must generate sufficient pore pressure rise and resulting brittle behaviour. Whether strain softening or limited strain softening response occurs depends upon the initial state of the tailings and the size and nature of the seismic event. The screening of site susceptibility to seismic induced liquefaction and subsequent tailings impoundment failure can be carried out using simple tools like those offered by Figure 9.4. Figure 9.4 separates operating from non-operating impoundments though the ability to judge future performance of a closed facility from the historical trend of “better” response for non-operating facilities may need to be discontinued as more facilities are being closed under saturated/flooded conditions. The more typical manner to evaluate the potential for
Figure 9.3  Simplified Characteristic Soil Strain Responses to Shear Stress Loading (adapted from Robertson and Fear, 1995)
Figure 9.4  Sample Empirical Screening Tool for Site Specific Liquefaction Susceptibility (adapted from Conlin, 1987)
seismic liquefaction is carried out using empirical relationships from some form of field density data. More fundamental evaluations using laboratory data coupled with an appropriate numerical tool can also be used and, as better numerical models come available, this approach is gaining popularity. The Author does not consider it to be prudent practice to use the latter approach without some form of in-situ data although this is a choice for the professional designer.

Static liquefaction is often a more difficult phenomenon to describe and/or anticipate than its seismic counterpart. There is limited mention of static liquefaction as a phenomenon in governance literature. As with seismic liquefaction, the most common manner to address static liquefaction design issues is to use empirical relationships. For example, Plewes et al. (1988) note approximate rates and construction lift thicknesses required to initiate a static liquefaction event in mine tailings deposited in BBW conditions. These approximate rates for saturated tailings are in the order of 5 m/day or higher.

To demonstrate whether a static liquefaction trigger exists for a given tailings facility, a generic slope configuration(s) and probable stress loading paths for the tailings deposit should first be appreciated. Figure 9.5 presents such a generic slope. The value of “S” is often termed the slope of the tailings facility (or as the horizontal component in H:V ratios). Both compressive and extensive stress path triaxial data are required to do a specific evaluation. From this data, at least conceptually the “collapse surface” can be approximately located within the lines of phase transformation (steady state) as shown on Figure 9.6. In practice, it is often difficult to establish a collapse surface in compressive loading. The lines of phase transformation are
NOTES:
1. SCHEMATIC ONLY
2. PASSIVE WEDGE AT TOE IGNORED (TRIAXIAL)
3. STRAIN LEVELS NOT IMPLIED
   (E.G. CAN VARY ALONG PLANES)
4. S=HORIZONTAL DISTANCE NORMALIZED SLOPE
   VALUE RELATIVE TO A UNIT OF VERTICAL SLOPE

Figure 9.5  Simplified Tailings Impoundment Stress States
Figure 9.6  Tailings Impoundment Slopes in Relation to Example Phase Transformation and Anisotropic Collapse Surfaces
identical in either compressive or extensive stress space. However, this isotropy is not evident with the collapse surface that is steeper in compressive loading than in extension. This anisotropy is largely due to fabric/grain imbrication (almost always preferential to the horizontal plane) and increases with grain angularity and elongation; two characteristics common to ground mill tailings. The imbrication, due mainly to the hydraulic deposition processes, results in elongated grains being aligned preferentially in horizontal to sub-horizontal layers. Typically, this horizontal plane is normal to the maximum principle stress resulting in additional cross-plane anisotropy in triaxial loading conditions.

Also shown on Figure 9.6 are equivalent tailings embankment slope values “S”. As a general rule, values of “S” must be greater than the collapse surface value to undergo “spontaneous” liquefaction; that is to experience a brittle liquefaction event with essentially negligible trigger application. For many tailings, a compressive loading slope of about 2H:1V to 3H:1V and an extensive loading slope of about 3.2H:1V to 3.9H:1V results; the implication being slopes flatter than these values do not have enough shear bias to be at risk of large-scale spontaneous/static liquefaction even under the loose states. It is interesting to note from the international database (e.g., ICOLD, 1994) that no tailings impoundment of either overall or large intermediate slope flatter than 4 to 5H:1V has failure attributable to a static liquefaction event. On the other hand, there are several upstream constructed tailings impoundments of overall or intermediate 2H:1V to 3H:1V slopes which are considered to have failed in this manner. Kramer and Seed (1988) demonstrated in the laboratory that there is a marked increase in static liquefaction susceptibility with increase in principal effective stress ratio. This type of soil behaviour has been observed by many other researchers and described in literature at least as far back as Bjerrum et al. (1961).
Another manner of looking at Figure 9.6 is to see how "far", in terms of slope, a given geometry is away from a static liquefaction concern. Combining Figure 9.6 with a fundamental understanding of the strain required to develop the indicated states can put the concern for a given tailings facility into perspective. Ignoring both loading rate, which must be great enough to raise pore pressures to near total stress levels in spite of drainage, and the unlikelihood of extensive, and contiguous areas of highly contractile tailings, differential shear strains in the order of 5% to 10% are typically required to induce static liquefaction. To have any appreciable thickness of tailings liquefy, equivalent rapid movement of 10's of centimetres to metres in tailings mass or in the foundation is usually required.

9.5 Assessing Liquefaction Susceptibility

9.5.1 General

Likely no other area of the geotechnical behaviour of mine tailings and/or hydraulic fill has received as much attention as the liquefaction susceptibility of these materials due to transient loading conditions. Seed (1987) provides a good overview summary of "founding" key literature defining the phenomenon. As typically occurs when considerable attention is given to any technical issue, both valuable and questionable literature is produced on the subject. The application of laboratory, field and in-situ testing methods to liquefaction susceptibility has not been immune to this trend and the literature is full of conflicting and contrasting views on how to best determine liquefaction susceptibility.

To simplify the issue for the assessment of mine tailings, liquefaction susceptibility assessment methods can most easily be divided into two distinct classes:
• empirically-based "direct" relationships; and,

• state-based "indirect" relationships.

The empirically-based "direct" relationships relate an in-situ measured indicator of density, often by an equivalent SPT "N" value, to past performance of similar ground during earthquakes. The method therefore requires both an assessment of density through some empirical technique and an estimate of ground shaking by deterministic or probabilistic methods. The overall uncertainty associated with combining these two variables, which each have potentially large systematic errors, can erode the confidence in such approaches. However, these empirical methods remain popular and most in-situ tests have published relationships to determine liquefaction susceptibility. These methods have also shown good practical success in design applications.

The "indirect" state-based relationships, on the other hand, determine in-situ density alone with no reference to a triggering mechanism. The intent of these relationships is to determine whether dilatant or contractant behaviour would occur upon shear straining; irrespective of the source of this shear loading. This determination has, however, often required calibrative laboratory testing to determine critical void ratios at a range of operative stress levels and material gradations.

As a complement to either the direct or indirect approaches, laboratory testing to determine cyclic or static liquefaction resistance is common engineering practice. However for cohesionless soils such as many mine tailings, the testing is typically carried out on reconstituted samples. Although these samples may approximate in-situ density, critical aspects of the material such as fabric, state-of-stress etc. can often not be adequately reproduced.
A significant potential advancement in laboratory testing of cohesionless soils comes from frozen sampling and controlled thawing where purportedly quite good approximations of in-situ character (other than state-of-stress) can be preserved. Pioneering work by the University of Alberta at B.C.'s Duncan Dam and at various locations, including the Mildred Lake Settling Basin as part of the CANLEX project, shows the potential for this method.

This section of the thesis summarizes the various methods of using piezocone data to assess liquefaction susceptibility.

9.5.2 Traditional SPT Approach

Simplified procedures to assess the potential for soil liquefaction during earthquake shaking were originally developed over the late 1960s and early 1970s. The genesis of many of these methods can be attributed to the late Professor Seed and his co-workers at the University of California at Berkeley. Seed's simplified liquefaction assessment charts (SLAC) were developed using measured SPT blowcounts in soils which did and did not liquefy in earthquakes from selected areas around the world; primarily California, China and Japan. The basis for using the SLAC is to calculate the Factor of Safety against soil liquefaction, $F_{SL}$, which is defined as:

$$ F_{SL} = \frac{CRR}{CSR} $$

(9.3)

where:

$CSR = \text{the average cyclic shear stress ratio induced in the soil during earthquake shaking}; \text{ and}$
CRR = the average induced cyclic shear stress ratio required to cause the onset of liquefaction, or cyclic resistance ratio.

FS_L greater than 1.3 is normally assumed to be required to prevent the onset of full soil liquefaction and limit the development of residual excess pore pressures, which can cause deformations, to acceptable levels.

The CSR induced in the tailings dam can be determined either by dynamic stress analyses or simplified procedures based on the more rigorous stress analyses. According to Seed's simplified method, the CSR in level ground cases can be calculated as:

\[
CSR = 0.65 \, a_{\text{max}} \frac{\sigma_v}{\sigma_v'} \, r_d
\]

where:
- \(a_{\text{max}}\) = peak horizontal ground acceleration at ground surface;
- \(\sigma_v\) = total vertical stress;
- \(\sigma_v'\) = effective vertical stress; and
- \(r_d\) = an empirical stress reduction factor.

For sloping tailings storage facilities, values of \(a_{\text{max}}\) represent the acceleration on the surface of the slope. Measurements of the acceleration of the crest of impoundments and dams during earthquakes show that the acceleration in the free field at the toe of the deposit is amplified through the body of the tailings deposit by a factor between about 1 and 4. The level of amplification depends upon the height of the deposit, slope of the deposit, tailings stiffness and earthquake magnitude.
The CRR can be determined from direct laboratory testing on representative samples, penetration resistance or other empirical method. The values obtained empirically from SPT \((N_1)_{60}\) penetration blowcounts, using methods such as described by Seed (1987), remains the most popular manner with which to estimate CRR. The database for the SPT was historically the largest of any available test method for empirically determining liquefaction susceptibility. Recent years has seen the piezocone becoming roughly equivalent in terms of case history data. The use of the SPT database requires determining a direct or equivalent \(N\) value (corrected for fines content and stress level) and estimating the cyclic stress ratio that will be induced by the earthquake as shown by Equation 9.4.

Fear and McRoberts (1995) reviewed the SLAC database and suggested upper and lower bounds for cohesionless soils with and without fines. This reinterpretation, and the original Berkeley interpretation for various levels of fines content, is shown on Figure 9.7. To use piezocone data on mine tailings to obtain an equivalent \((N_1)_{60}\) for use with Figure 9.7, Equation 8.2 can be modified to be:

\[
(N_1)_{60} = C_N \times q_c \text{ (MPa)} / [0.85 (1 - L/4.75)]
\]

(9.5)

where: \(C_N = 2/(1 + \sigma'_v)\) for \(\sigma'_v < 100\) kPa (Skempton, 1986)
or \((1/\sigma'_v)^{0.56}\) for \(\sigma'_v \geq 100\) kPa (Jamiolkowski et al., 1986)
Figure 9.7 SPT Liquefaction Susceptibility Chart (adapted from Fear and McRoberts, 1995)
9.5.3 Piezocone Tip Stress Approaches

The two main piezocone tip stress approaches, so named as they make use of only the tip stress in their liquefaction susceptibility assessments, are:

1. the \((Q_c)\) threshold approach such as that recommended by Sladen and Hewitt (1989); and,

2. the original CPT state parameter approach as presented by Been and Jefferies (1985).

The piezocone would appear to be well-suited to empirical liquefaction susceptibility assessments due to the high strain nature of the test and its ability to distinguish material type, relative compressibility and in-situ density. However, traditional tip stress approaches such as the two noted above have not worked well in some cases. The problem with these traditional approaches is that piezocone tip-resistance alone, either for a Been and Jefferies state parameter, an empirical Seed-type evaluation or a Sladen and Hewitt (1989) \(Q_c\), appears to be restricted to uniformly-graded sands as an indicator of liquefaction susceptibility.

The original state-parameter and \(Q_{c1}\) approaches both have received deserved negative appraisal due to a lack of consideration for, among other things, material compressibility. Continued adherence to a piezocone tip stress value alone for liquefaction susceptibility may not lead to excessive error if clean sand tailings only are evaluated, although such sites in practice are certainly in the minority. Even for relatively coarse-grained oil-sand tailings such as at Syncrude or Suncor, materials often considered uniformly graded, it will be shown later that use of such tip stress approaches can lead to inaccurate liquefaction susceptibility assessments. Examples later
in this chapter will also show these methods can lead to both conservative and non-conservative evaluations of liquefaction susceptibility.

### 9.5.4 Shear Wave Velocity Approaches

The use of small strain ($\sim <10^{-6}$) shear wave velocity has been postulated with several methods as a potential indicator for liquefaction susceptibility. As most current piezocones allow shear wave velocity profiling with little additional effort during a site characterization program, a review of shear wave techniques is warranted. The three main postulated approaches are:

- an empirical approach similar to the SPT database method (e.g., Stokoe, 1990; Robertson, 1990);
- an empirical approach whereby equivalent SPT-N or CPT-Q values are predicted from the shear wave velocity and then SLAC evaluations are carried out; and,
- an empirical approach where void ratio and/or state parameter can be estimated by the shear wave velocity.

However, at the outset, the author contends that based upon fundamental soil mechanics and the tailings database, shear wave velocity would not appear to be a good indicator of liquefaction resistance. Shear wave velocity is indicative of skeletal stiffness at very low strain and is highly sensitive to aging, cementation, and disturbance. This sensitivity would seemingly render practical use of shear wave velocity for liquefaction susceptibility and related issues as being extremely limited in mine tailings. For further background, Roy et al. (1997) note the relatively poor correlation between small-strain shear wave velocity and large-strain soil behaviour.
From the tailings database, the Syncrude tailings data casts doubts on the ability of shear wave velocity to assess in-situ state. Figure 9.8 shows Syncrude data (Robertson, 1995) where normalized shear wave velocity has been compared with in-situ void ratio. This data is for one material; the coarse, non-crushed, young and normally consolidated Syncrude mine tailings. The material is likely as close as any in-situ tailings can be for lack of natural aging, cementation and other processes which have very large impact on measured shear wave velocity. Figure 9.8, shows a scatter from lower to upper bound in the order of 0.10 in void ratio space for this favourable correlative sand. The maximum and minimum void ratios for Syncrude tailings, determined in accordance with ASTM D4353-91 and D4254-91, are 0.893 and 0.522 respectively (Vaid et al., 1995). Consequently, for a given value of normalized shear wave velocity for this Syncrude sand, the minimum scatter error is over 25% in terms of relative density. Furthermore, the flatness of the relationship shown in Figure 9.8 implies extreme void ratio sensitivity with relatively minor changes in shear wave velocity (e.g., 10 m/s is roughly equivalent to 0.05 in void ratio). The roughly 25% scatter error and the sensitivity of measurement concern is for a single sand; universal relationships would be expected to be considerably poorer.

The situation is not improved when attempts are made to correlate stress-normalized shear wave velocity, \( V_{s1} \), to piezocone tip stress. Figure 9.9 shows published data from the Fraser Delta in British Columbia (Robertson, 1995). On Figure 9.9, the correlative trend line between tip stress and shear wave velocity is presented as suggested by Robertson (1995) and Cunning et al. (1995). Given the data dispersion, even in log-log space, it can be argued that there is invariance between \( V_{s1} \) and piezocone tip stress for this data set. The importance of this particular data set
\[ V_{s1} = V_s \times (100/p'_c)^{0.26} \]

**Figure 9.8** Shear Wave Velocity versus Void Ratio - Syncrude Tailings
(adapted from Cunning et al, 1995)
Figure 9.9  Normalized Shear Wave Velocity versus Cone Tip Stress - Lower Mainland Deltaic Data (adapted from Cunning et al, 1995)
is that it forms the basis for the current literature promoting the shear wave velocity approaches (e.g. Cunning et al., 1995). Criticism of trying to draw a correlation between Vs1 and Qc1 from the data in Figure 9.9 is not a particular condemnation of that data set. Other investigators (e.g. Rix and Stokoe, 1988) have also found similar problems in correlating shear wave velocity with the large strain geotechnical behaviour. The lack of corroboration between Vs1 and Qc1 should not be viewed as surprising.

Figure 9.10 shows data from the Sullivan mine tailings facility in an area adjacent to where a static liquefaction flow event occurred in 1991 (described later in this chapter and in Davies et al., 1998). The liquefied tailings embankment was roughly 15 metres in height, and was constructed by upstream methods with an overall slope in the order of 3H to 1V. Sample shear wave velocity data from the failure area at Sullivan Mine is shown on Figure 9.10. Also shown on Figure 9.10 are both the lower (Alaskan) and upper (Syncrude) bound lines for state parameter \( \psi = 0 \) (neutral static liquefaction resistance) proposed by Cunning et al. (1995). Cunning et al. (1995) suggest that their lower and upper bounds should encompass most geologic materials. A non-conservative conclusion would have been made at this base metal mine if the shear wave velocity approach suggested by Cunning et al (1995) has been used for design. The tailings at the Sullivan mine have compressibilities more similar to the Alaskan tailings than to Syncrude tailings.
SHEAR WAVE VELOCITY (m/s)

**Figure 9.10** Shear Wave Velocity profile at Sullivan Mine Liquefaction Area versus Published Neutral-State Relationships
Overall, shear wave velocity approaches likely do not work well for liquefaction screening of mine tailings due to most mine tailings deposits, as opposed to many most natural sediments, having some degree of interparticulate bonding not related to strength. At the Sullivan Mine and at many other tailings sites, these bonds may be partially due to geochemical alteration (e.g., sulphide mineral oxidation processes). This bonding which exists at very low strains (e.g., seismic test) is eliminated upon any appreciable shearing and resulting large strain behaviour, such as during a liquefaction event, and therefore cannot be confidently or consistently anticipated. Figure 9.11 shows a simple schematic comparing two units of tailings following sampling or testing disturbance. The second unit will exhibit higher apparent low strain “stiffness” although these chemical bonds readily break down under any appreciable shear strain. Consequently, shear wave velocity is not considered a viable predictive vehicle for estimating the large strain behaviour of tailings where geochemical bondings may be quite common and is therefore not recommended for any large-strain evaluative work at mine tailings facilities. However, in addition to the reasons cited by the Author and his colleagues in Roy et al. (1997), shear wave velocity methods may have applicability in uncemented and normally consolidated sands.

9.5.5 Non-Piezcone In-Situ Methods

There are several other in-situ methods that have been postulated for assessing liquefaction susceptibility. Other than the self-boring pressuremeter, only in-situ geophysical methods, using either nuclear or resistivity logging tools, appear to have any practical validity. However, the geophysical logging techniques have not been shown to be any more accurate than other in-situ methods (e.g., Sobkowicz and Handford, 1990). A detailed evaluation of resistivity logging
Figure 9.11 Schematic of Influence Cementation Bonds in Tailings have on In-Situ Characteristics
techniques in tailings at Syncrude's Mildred Lake Settling Basin showed a standard deviation of between 0.09 and 0.20 in void ratio space (Sobkowicz and Handford, 1990). This level of uncertainty is at least as poor as that determined by the non-recommended shear wave approaches. Pressuremeter methods were not addressed by this research.

9.5.6 Integrated Piezocone Approach

The state parameter approach using piezocone data developed in the mid 1980's (e.g., Been and Jefferies, 1985) has been upgraded as described in Chapter 7. The suggested manner to use piezocone data for state-based liquefaction susceptibility evaluations can now be carried out in a framework which includes material compressibility. This inclusion is very important for materials of variable compressibility; a class to which mine tailings certainly belong. The recommended piezocone-\(\psi\) relationships by Been and Jefferies (1985) neglected material compressibility, \(\lambda\). The main problem in the original work was the limited gradation range of materials tested.

It could be argued that the work of Been and Jefferies (1985) and Been et al. (1987) provided a way of interpreting piezocone data for different cohesionless soils, including tailings. However, many processed (e.g., tailings) and natural sands contain appreciable amounts of finer materials than the sands tested for the original approaches. The state parameter method for the piezocone interpretation presented in Chapter 7 now includes not only sandy tailings but tailings classifying as sandy silts, silts and even clays using the observation of good correlation between piezocone penetration resistance and overconsolidation ratio (Wroth, 1988; Sills et al., 1988; Crooks et al., 1988). It is suggested that this proposed “Integrated” approach represents a systematic method
for estimating the initial state of mine tailings. Site specific liquefaction susceptibility screening of tailings can be carried out by evaluating the piezocone data within the contours of \( \psi \) as shown on Figure 7.5 in Chapter 7.

Figure 7.5 can also be used to show why, as discussed in Section 9.5.3, approaches using tip stress alone may lead to either conservative or non-conservative results. For example, the Ukalerk and Nerkerk piezocone data presented by Sladen and Hewitt (1989) was obtained in material with ranges of piezocone friction ratios of about 0.1% to 0.9% with a predominance around 0.1% to 0.2%. Using Figure 7.5, the "critical" value of Q where \( \psi = 0 \) is shown to range from about 15 to nearly 200; a factor of more than an order of magnitude. The recommended ratio of mean piezocone tip stress to mean effective stress recommended by Sladen and Hewitt (1988) was about 80 which is roughly equivalent to a critical Q value for a friction ratio of 0.15%. Consequently, the proposed critical \( q_c' / \sigma_v' \) method by Sladen and Hewitt (1986) would be quite good for materials at about \( F = 0.15\% \) but overly conservative for higher values and potentially non-conservative for the coarser, less-compressible sand fills. The disagreement in literature regarding the state of Beaufort Sea hydraulic fills, as determined from piezocone data, was largely a function of not appreciating this compressibility effect, and/or the role that compressible fines can play in liquefaction susceptibility.

9.6 Sample Liquefaction Susceptibility Screening - Integrated Piezocone Approach

9.6.1 Overview

As noted earlier, the Integrated Piezocone Approach introduced in Chapter 7 was applied to all of the piezocone soundings obtained during the research. The algorithm provided by either
Equation 7.8 or 7.9 is readily automated into piezocone interpretation software (e.g. presently in research versions of CPTINT) or, as was used for the majority of this research, in commercial spreadsheet applications. The keys to obtaining the most representative results include:

- use of the correct groundwater table in the calculations (Chapter 10);
- inclusion of in-situ vertical gradients as these can greatly influence computed stress level (Chapter 10); and
- use of reasonable unit weight values for the tailings materials.

The latter point is important for many mine tailings as the typical invariance in dry unit weights for most natural geotechnical materials do not necessary hold for mine tailings. For example, Table 9.2 shows some typical values for the specific gravity of some typical tailings materials.

<table>
<thead>
<tr>
<th>Tailings Material</th>
<th>Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Copper</td>
<td>2.7</td>
</tr>
<tr>
<td>Carbonate Copper - Zinc</td>
<td>2.9</td>
</tr>
<tr>
<td>Siliceous Copper - Zinc</td>
<td>4.0</td>
</tr>
<tr>
<td>Oil Sand</td>
<td>2.7</td>
</tr>
<tr>
<td>Molybdenum</td>
<td>2.7 - 2.8</td>
</tr>
<tr>
<td>Gypsum</td>
<td>2.4</td>
</tr>
<tr>
<td>Iron (Base Metal)</td>
<td>4.2</td>
</tr>
<tr>
<td>Silica (Base Metal)</td>
<td>3.3</td>
</tr>
<tr>
<td>Bauxite</td>
<td>2.8 - 3.3</td>
</tr>
<tr>
<td>Gold</td>
<td>2.6 - 2.7</td>
</tr>
<tr>
<td>Tropical Lead - Zinc</td>
<td>2.9 - 3.0</td>
</tr>
<tr>
<td>Temperate Lead - Zinc</td>
<td>3.5</td>
</tr>
<tr>
<td>Fine Coal</td>
<td>1.5</td>
</tr>
</tbody>
</table>

*Values obtained from Vick (1990) and the Author's files.
The values in Table 9.2 are for illustrative purposes only. For any significant evaluation of mine tailings for liquefaction susceptibility, site specific assessment of specific gravity is important. Following are several examples where the Integrated Piezocone Approach is applied to assess liquefaction susceptibility. In each case, best attempts were made to correctly characterize in-situ pore pressure and unit weight conditions. Per Chapter 7, static liquefaction is considered possible for $\psi$ values of roughly zero and higher whereas seismic liquefaction can be a concern right up to $\psi = -0.1$. From the Author's experience, a margin of error of +/- 0.03 on the state developed from the piezocone is considered prudent prior to declaring that a material is or is not potentially susceptible to liquefaction.

### 9.6.2 Beaufort Sea Hydraulic Fills

As was noted earlier in this Chapter, the piezocone assessment of hydraulic fill density work in the Beaufort Sea may have been originally inappropriately carried out due to a lack of an assessment framework such as described for the integrated piezocone approach. Considerable practitioner and academic debate on the in-situ state of the Beaufort Sea hydraulic fill used in platform construction resulted due to failures which occurred in these fills. The discussions included assessment of the materials which did liquefy and of the large piezocone database which indicated a wide variety of conclusions not readily consistent with the actual performance of the fills.

A number of Beaufort piezocone profiles were assessed with the Integrated Piezocone Approach and the method by Been et al. (1987) which was used as the basis for much of the published
Beaufort interpretation of in-situ state. Figure 9.12 presents a sample evaluation using two piezocone traces from the Amauligak core verification work.

The trace on the left of Figure 9.12 shows good agreement between the Integrated Piezocone Approach and the Been et al. (1987) approach of assessing in-situ state from piezocone data. At this piezocone sounding location, the average friction ratio, F, was about 0.4% with little variation. On the right hand side of Figure 9.12, another comparison between the Integrated Piezocone Approach and Been et al. (1987) is shown. Here, the agreement is fairly good in terms of trend but a shift of about +0.04 to +0.06 is relatively consistent throughout the profile. The material at this latter piezocone sounding had F values closer to 0.2% and, as can be shown by Figure 9.12, this correction for F would explain the shift in assessed state. The concern with the latter comparison is that the method Been et al. (1987) was non-conservative. The resulting error in \( \psi \), in not taking material compressibility (index) into account, is roughly equal to 0.05; the approximate difference in void ratio argued by the profession at the time of the Nerlerk berm failure.

9.6.3 Sullivan Tailings

As will be described later in this Chapter, the Sullivan Mine experienced a static liquefaction failure and flow of a portion of its Active Iron Pond Dyke in 1991. There was also a similar failure at the mine in 1948 in another area of the overall tailings complex. The 1948 flow failure, which traveled for several kilometers, occurred in materials similar to those involved in the 1991 event.
Figure 9.12 Comparison of Original State Parameter and Integrated Piezocone Approaches - Beaufort Sea Hydraulic Fill Data
Over the course of the research, there were a number of piezocone soundings obtained around the vicinity of both the 1991 and 1948 liquefaction slump events. Figures 9.13 to 9.16, inclusive, show two such soundings interpreted with the Integrated Piezocone Approach as follows:

- Figures 9.13 and 9.14 - immediately adjacent to the 1991 liquefaction slump (east of area);
- Figures 9.15 and 9.16 - at the approximate backscarp of the 1948 failure.

In each case, two figures are used for each sounding to show a basic classification of points (showing the degree of potentially contractant materials in the given sounding) and the far more useful plot of state versus depth. This latter plot can be used to plan more detailed sampling programs, determine the degree of material to assign potentially brittle behavior to for numerical analyses or any other similar engineering endeavor.

It is clear from these few sample soundings from the Sullivan Iron Tailings that liquefaction susceptibility under static loading conditions for some tailings layers would have been well predicted by the Integrated Approach. As noted previously in this Chapter, the shear wave velocity data from the seismic piezocone soundings was not successful in doing the same.

9.6.4 Alaskan Tailings

Cunning et al. (1995) describe a carbonate-rich mine tailings from Alaska which they claim were difficult to classify for liquefaction susceptibility due to the high degree of compressibility in the tailings. Cunning et al. (1995) also show how the piezocone tip stress method proposed by Robertson et al. (1992) would have resulted in the highly conservative conclusion that most of the tailings were in a contractant state.
Figure 9.13  Integrated Piezocone Approach - Sullivan SV4-9518 (Classification Plot)
Adjacent to 1991 Liquefaction Slump
Figure 9.14: Integrated Piezocone Approach - Sullivan Mine SV4-9518 (Depth Screening Plot) Adjacent to 1991 Liquefaction Slump
Figure 9.15  Integrated Piezocone Approach - Sullivan SV10-9608 (Classification Plot) near Backscarp of 1948 Liquefaction Failure
Figure 9.16 Integrated Piezocone Approach - Sullivan SV10-9808 (Depth Screening Plot) near Backscarp of 1948 Liquefaction Event
Figure 9.17 shows the Integrated Piezocone Approach interpretation of a piezocone sounding from the carbonate-rich Alaskan tailings. The Alaskan tailings data was provided by Robertson (1995, personal communication). Clearly, the conclusion from using the Integrated Approach would be that the tailings were in a relatively non-strain softening state which agrees with the conclusions for this material made by Cunning et al. (1995) based on field and laboratory testing. In other words, the piezocone could predict the results of a detailed assessment of liquefaction susceptibility. However, the tip-stress approach was not valid.

9.6.5 Mildred Lake Settling Basin

The Mildred Lake Settling Basin provides an excellent test of the Integrated Piezocone Approach. The first example is from Cell 24 where the detailed characterization work for Phase I of the CANLEX project was carried out. Figure 9.18 shows the interpreted in-situ state from the piezocone data. The values from 33 to 45 metres depth, all roughly $\psi = -0.08$ to -0.12, represent the CANLEX "target zone" data. These values correspond to equivalent void ratios (relative to laboratory established critical state) entirely consistent with values determined for the same zone using geophysical techniques, (Küpper et al., 1995) and frozen samples (Hoffman, 1997).

Another example of the application of the Integrated Approach is provided by piezocone data obtained along the entire length of a hydraulically deposited tailings beach created during placement of the Cell 4 to 10 toe berm at the Mildred Lake Settling Basin. This example is used to show how the approach can differentiate between soils deposited in states ranging from loose
ALASKAN TAILINGS SANDS (ROBERTSON, PERS. COMM.)

Figure 9.17. Integrated Piezocone Approach - Alaskan Tailings
Figure 9.18  Integrated Piezocone Approach - CANLEX Mildred Lake Tailings
to dense and where material gradation range could be expected to be close to maximum sizes for Syncrude tailings. Figure 7.10 showed that range of CPT data obtained in the upper beach zone near the point of tailings discharge, middle beach zone and the lower beach near spillbox collection areas. The lower beach data includes tailings deposited below water near a spillbox.

9.6.6 J-Pit CANLEX Trial

As noted in Section 5.2, there was an attempt to trigger a full-scale field liquefaction event using BBW tailings deposited in Syncrude's J-Pit. The event did not create a significant flow slide event as predicted by many of the CANLEX participants. Perhaps the most critical aspect of the J-Pit liquefaction field trial is that the BBW tailings may likely not have been in a contractant in-situ state at the stress conditions present. Figure 9.19 to 9.21, inclusive, show three of five CPT cross-sections through the J-Pit data with interpreted state by the Integrated Piezocone Approach. The location of the sections is shown in Appendix I. A minority percentage, perhaps as low as 5%, of the overall deposit, based upon these results, showed a positive (contractant) state. From the point-of-view of the Integrated Approach, the resulting behaviour of the trial was not unexpected; it was predicted prior to the event.

9.6.7 Syncrude SWSS Facility

More than 125 piezocone soundings were carried out at Syncrude Canada Ltd.'s SWSS Facility as part of a comprehensive program to deduce the deposited state of the sand tailings. Each of these piezocone sounding was evaluated using the Integrated Piezocone Approach as a means of carrying out a comprehensive liquefaction susceptiblility screening of the SWSS Facility.
Figure 9.19  Integrated Piezocone Approach - Syncrude J-Pit CANLEX Line 2
From a review of all of the SWSS Facility piezocone data, compiled in a form similar to Figure 9.22, conclusions can be drawn from the screening results which are consistent with observed, modeled and laboratory tested behaviour:

- the large majority of BAW controlled zone tailings are strongly dilatant;
- occasional thin zones of positive state material exists in the BAW tailings but these are isolated, small percentage volume occurrences; and,
- roughly one half of BBW tailings generally exist in either a neutral or positive state indicating similar results to the toe berm piezocone data from the Mildred Lake Settling Basin shown in Chapter 7.

The ease of applying the Integrated Piezocone Approach and the visual clarity of plots such as Figure 9.22 (at appropriate scale) make screening in-situ state a very straightforward exercise. Note that Figure 9.22 is provided for relative illustrative use only, to provide an example of how the Integrated Approach can be summarized onto cross-sections.

9.7 Operative Undrained Strength

9.7.1 Overview

Selecting an undrained strength, steady-state strength, residual strength or whatever the selected term for liquefied tailings can be difficult. The difficulty mainly arises from the manner in which the engineering profession has addressed this selection process.

Undrained strength of tailings, which usually represents the “critical” or “minimum assured” strength condition, was initially thought to be negligible and liquefied soils were attributed zero shear strength under these conditions. For many engineering applications, the assumption of
zero strength is unduly conservative and ignores field and laboratory performance data. In the latter part of the 1970's, the profession began to appreciate that cohesionless materials could indeed have undrained strength. Discrepancies between field and laboratory studies (the field residual versus laboratory steady-state strength issue), however, led to uncertainty in practice and considerable practical and academic debate about how to address the undrained strength issue.

A further problem has been a lack of distinction between what are fundamentally two different shear strengths:

1. The undrained strength which develops under constrained conditions with moderate but limited strain following a liquefaction event. This strength can be used in limit-equilibrium assessments to determine post-liquefaction stability; and,

2. The undrained strength resulting from almost unlimited strained material involved in unconstrained liquefaction flowslide events.

The first strength is applicable to tailings impoundment stability evaluations and would be non-conservative for runout purposes. Furthermore, as discussed further below, this "constrained" undrained strength is directly a function of initial stress state. The second strength has no stress-history influence and is purely a function of initial density. This unlimited strain undrained strength is an overly conservative value to be used in many stability assessments including the evaluation of post-liquefaction stability for most tailings facilities (e.g. Lo and Klohn, 1991, McLeod et al, 1991, and Pilai and Stewart, 1993).
9.7.2 Stress Level Dependent - Constrained Analyses

The original database by Seed (1987), and updated by Seed and Harder (1990), is shown in Figure 9.23. The form of the relationship offered by Figure 9.23 included essentially no strength allowance for materials with SPT blowcounts below 4 with no attributable strengths above 800 to 1000 psf (40 to 50 kPa). For tailings facilities of any appreciable height, the implied undrained strength limitations of Figure 9.23 are quite severe as shown later in this section. Many existing structures are shown to be wholly inadequate to support themselves should liquefaction occur and the undrained strengths predicted by Figure 9.23 manifest themselves. This condition of severity is particularly true for structures over 30 m in height. It is important to note the strong bias towards low stress level conditions in the data set shown in Figure 9.23.

In the latter part of the 1980’s, evaluations of both laboratory and field data tended to indicate undrained strength of tailings and other hydraulic fills was a function of initial confining stress. Although this stress normalization did not remove all of the problems with differing literature approaches to that time, the majority of published literature fit this approach quite well. The Author (McLeod et al., 1991) published an initial effort on this concept which included a re-evaluation of the liquefaction case histories used by Seed (1987), Poulos et al. (1985) and Seed and Harder (1990). Table 9.3 presents the re-evaluated data. In Table 9.3, the recommended value of $\sigma_{vo}'$ is the initial vertical effective stress at the mid-point of the liquefied zone.
Figure 9.23  Predicted Undrained Residual Strength from SPT $(N_1)_0$ (adapted from Seed and Harder, 1990)
Table 9.3 Undrained Residual Strength Data for Several Case Histories

<table>
<thead>
<tr>
<th>Case History</th>
<th>Undrained Strength, $S_u$ (kPa)</th>
<th>Equivalent Clean Sand SPT $N_i$ at 60% of $S_u$ (kPa)</th>
<th>Initial Vertical Confining Stress, $\sigma_{vo}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Fort Peck</td>
<td>17 - 34</td>
<td>10 - 11</td>
<td>290</td>
</tr>
<tr>
<td>2. Heber Road</td>
<td>5</td>
<td>1 - 2</td>
<td>$\approx$ 35</td>
</tr>
<tr>
<td>3. Juvenille Hall</td>
<td>6 - 10</td>
<td>4 - 11</td>
<td>$\approx$ 40 - 120</td>
</tr>
<tr>
<td>4. Kawagishi</td>
<td>6</td>
<td>4</td>
<td>$\approx$ 30</td>
</tr>
<tr>
<td>5. Koda numa</td>
<td>2 - 3</td>
<td>3</td>
<td>$\approx$ 25</td>
</tr>
<tr>
<td>6. La Marquesa (Downstream)</td>
<td>19</td>
<td>11</td>
<td>100</td>
</tr>
<tr>
<td>7. La Marquesa (Upstream)</td>
<td>10</td>
<td>6</td>
<td>57</td>
</tr>
<tr>
<td>8. La Palma</td>
<td>10</td>
<td>4</td>
<td>62</td>
</tr>
<tr>
<td>9. Lower San Fernando Dam</td>
<td>19 - 36</td>
<td>8 - 15</td>
<td>192</td>
</tr>
<tr>
<td>10. Mochikoshi Tailings No.1</td>
<td>3 - 12</td>
<td>4 - 6</td>
<td>57</td>
</tr>
<tr>
<td>11. Sheffield Dam</td>
<td>2 - 4</td>
<td>6</td>
<td>57</td>
</tr>
<tr>
<td>12. Soltafara Canal</td>
<td>2 - 6</td>
<td>4 - 5</td>
<td>$\approx$ 20</td>
</tr>
</tbody>
</table>

Figure 9.4 shows an adaptation of the re-evaluated database from this initial effort. The trend from the re-evaluation in Figure 9.24 yields:

$$\frac{S_u}{\sigma_{vo}} = (A) \left( N_i \right)_{60}$$  \hspace{1cm} (9.6)

where:

$A$ = empirical constant with a range of 0.008 to 0.026

$(N_i)_{60}$ = energy corrected (60% theoretical maximum) SPT at 1 tsf stress level
Figure 9.24 Re-evaluation of Liquefaction Case Histories
About the same time as McLeod et al. (1991), others such as Lo and Klohn (1991) were arguing for a rationale to deal with the low strength values implied by the Seed database for practical design purposes. For example, Figure 9.25 after Lo and Klohn (1991) shows the trend of undrained residual strength with height of overburden; i.e. an additional implication of stress level dependence.

By way of the suggested trends from Figures 9.24 and 9.25, which are formulated with the same case histories as Seed (1987) and Seed and Harder (1990), several researchers at essentially the same time (McLeod et al., 1991, Been et al., 1991, and Lo and Klohn, 1991) showed that the database itself suggested that confining stress level was a factor in the constrained undrained strength. Consequently, it was postulated that the $S_u/\sigma'_v$ approach could possibly be an appropriate manner in which to assess the selection of some design strengths. Following the original work cited above, others have followed this line of thinking and the literature has presented several examples using the $S_u/\sigma'_v$ approach such as Stark and Mesri (1992) and Pilai and Stewart (1993). The reference by Stark and Mesri (1992) is summarized by Figure 9.26. The work by Stark and Mesri is interesting as it includes both the field database re-examined by the Author and a considerable amount of laboratory data. Although there are a wide range of stress paths involved in the laboratory data in Figure 9.26, the contribution provides general confirmation of the trends noted in Equation 9.6.

The practical importance of determining whether a stress-normalization approach can be applied or not is readily apparent when viewed in terms of Figure 9.27. Here two assessments of undrained strength for two example in-situ densities ($\langle N_1 \rangle_60 = 4$ and 10) are plotted using a
Figure 9.25  Possible Relationship between Residual Strength and Height of Overburden (adapted from Lo and Klohn, 1991)
EQUIVALENT CLEAN SAND SPT BLOWCOUNT

Figure 9.26  \( S_u/p' \) versus \((N_1)_{60-ECS}\) (adopted from Stark and Mesri, 1992)
Figure 9.27  Comparing Stress Normalized and Non-Normalized Undrained Strength Relationships
median bound value from Figure 9.23 and then the lower and conservative upper bounds from Figure 9.24 and Equation 9.6. For a moderate height tailings dam, say 30 metres, the difference between even the lower bound stress normalized strength and the non-normalized value can be in the order of 3 to 5 times. For greater heights (confining pressure), the difference becomes considerably more. Clearly, designing to the requirements of Figure 9.23 for constrained analyses is a lot more restrictive than using, for example, Figures 9.24 or 9.26.

The empirical evidence summarized above indeed points to an $S_u/\sigma'_{vo}$ type of relationship existing for the liquefied strength of cohesionless soils for at least some situations. However, whether such a relationship would be predicted based upon more fundamental soil mechanics must be addressed. To help address this issue, some theoretical aspects and practical observations from field and laboratory evidence were examined during the research (Davies and Campanella, 1994) and are summarized below.

The stress level dependence of undrained strength can be examined using concepts of state in the simplified sense per Figure 7.1. To simplify the demonstrated relationship, three assumptions are made:

1. a unique critical state exists for a given soil;
2. that unique state can be described by a straight line in e-$\log_{10}(\sigma)$ space; and
3. the stress level invariance implied by the critical state model for a given in-situ void ratio is not necessarily supported by field evidence or arguments made in this thesis.
Assume that a given unit of tailings resides in-situ at an initial state loose of critical state with void ratio $= e_i$. Any shear would result in movement towards steady-state which requires either contraction of the skeleton ($\Delta e = e_i - e_{ss}$) or a reduction in effective stress due to pore pressure rise. The pore pressure rise occurs due to the shearing that is occurring under the constraint of zero volume change (undrained loading). For this loading, the following relation will hold:

$$\Psi = \lambda \log_{10} \left( \frac{p'}{p_{ss}} \right) \quad (9.7)$$

which, by rearrangement, leads to:

$$p_{ss} = p' 10^{(-\Psi/\lambda)} \quad (9.8)$$

Reverting to Mohr type strength definition where $S = \text{the strength of the material}$:

$$S = p'_{ss} \sin \phi_{ss} \quad (9.9)$$

or, taking $\phi_{ss} = 30^\circ$, for example,:

$$S = (0.5) p' 10^{(-\Psi/\lambda)} \quad (9.10)$$

Therefore, with the simplified example given, the strength is linearly related to the initial all around confining stress, $p'$ which, for convenience, can usually be considered to be directly proportional to $\sigma'_{vo}$. Moreover, the slope of the linear relationship will be a function of both initial density state ($\Psi$) and material type, gradation and fabric which is approximately expressed by the slope of the steady state line ($\lambda$). The influence of $\lambda$ in Equation 9.10 is discussed further below.
Another piece of evidence is available through analogy to plastic silts and clays. These finer-grained soils do have cohesion but undrained loading is less a function of cohesion as it is skeletal arrangement and other fabric related strength components. In cohesive soils, both peak and residual strength profiles show relatively linear undrained strength versus effective confining stress relationships. Without any theoretical appeal, one could surmise that undrained strength behaviour for particulate materials, like tailings, whether quasi-spherical or platy in shape, could well follow some unified construct.

Overall, some stress level dependence to undrained residual strength appears both “theoretically” and practically supported for constrained situations. However, to estimate such dependency by use of in-situ penetration resistance, either SPT or CPT as in Figures 9.24 and 9.26, does not quite capture all of the issue.

Penetration resistance of a given soil is highly influenced by the compressibility of the soil. Mine tailings are a good example of materials that have a wide range of material compressibility. As such, strength relationships based solely upon penetration resistance are incomplete. Figure 9.28 represents a compilation of several natural soils and mine tailings with fines content (defined as finer than 74 μm) ranging from 0 to 100%. Note that both the position of the steady-state line, or the relative value of Ψ for a given hydrostatic state, and the slope of the line, λ, are dependent upon fines content. The largest effect is on position but slope can also change by enough to warrant inclusion in a strength model.
Figure 9.28  Summary of Assorted Laboratory Steady-State Testing Data (adapted Davies and Campanella, 1994)
Figure 9.29 schematically represents an example from Davies and Campanella (1994) using a non-fines corrected penetration resistance versus a normalized strength. The information in Figure 9.29 is from mine tailings and was provided by Plewes (1994, personal communication). The data is from the Kennecott Magna tailings facility near Salt Lake City, Utah.

Figure 9.30 presents a summary of the recommended global relationship for estimating the undrained strength where density (state) and fines content information is available. The density values can take the values of \((N_1)_{60}\), state parameter \((\Psi)\), relative density \((D_r)\) or stress normalized CPT tip \((Q)\). Fines content can be a literal percentage or an empirical relation to \(I_c\) as described earlier in the thesis. Using \((N_1)_{60}\) (not fines corrected) and actual measured fines content, cut-off values of \(s_u/\sigma'_{vo}\) are 0.06 and 0.10 result which are dependent upon the fines content. From \((N_1)_{60}\) at 6 blows per 300 mm, each line or surface in the tripartite space follows a linear extension up to the drained strength which, for most materials, will be at about \(s_u/\sigma'_{vo} = 0.65\) to 0.70.

For the 30-60% fines material, which also represents the lower bound strength profile, the relationship per Table 9.4 results.
Figure 9.29  Example of Mine Tailings Data - Stress Normalized Undrained Strength
(adapted from Davies and Campanella, 1994)
*VALUES OF $(N_1)_{60}$ HAVE NO FINES CORRECTION

$A = 0\%$ to $30\%$ FINES AND $>60\%$ FINES

$B = \sim 30\%$ TO $45\%$ FINES

Figure 9.30  Schematic of Proposed Relationship for Estimating $S_u/\sigma_v$ for Mine Tailings
Table 9.4  Recommended Approach for Estimating Undrained Strength of Liquefied Tailings for Stability Assessments Using SPT Data (modified from Davies and Campanella, 1994)

<table>
<thead>
<tr>
<th>((N_i)_{60}) (blows per 300 mm)</th>
<th>LESS THAN 30% OR GREATER THAN 60% PASSING NO. 200 SCREEN, (s_d/\sigma_{vo})</th>
<th>BETWEEN 30% AND 60% PASSING NO. 200 SCREEN, (s_d/\sigma_{vo})</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 6</td>
<td>0.10</td>
<td>0.06</td>
</tr>
<tr>
<td>6 - 30</td>
<td>(0.10 + 0.025[(N_i)_{60} - 6])</td>
<td>(0.06 + 0.025[(N_i)_{60} - 6])</td>
</tr>
<tr>
<td>&gt;30</td>
<td>0.65 to 0.70</td>
<td>0.65 to 0.70</td>
</tr>
</tbody>
</table>

Where only piezocone data is available, procedures to determine both equivalent SPT and fines content values from \(I_c\) were provided earlier. In addition, by assuming some typical properties for mine tailings, Table 9.5 can be used for very quick estimates of the undrained strength ratio. Q values of 15 and 30 for silty and sandy tailings respectively are roughly equivalent to an \((N_i)_{60-ECS}\) value of approximately 10 blows per 300 mm. The approximate corresponding value of \((N_i)_{60-ECS}\) for Q equal to 45 and 70 for the silty and sandy tailings is 20 blows per 300 mm. Also included on Table 9.5 is an indication of the typical values of peak frictional resistance that can be used for static limit-equilibrium analyses.
### Table 9.5 Proposed Screening Level Estimation Relationships for Stress Normalized Undrained Residual Strength

<table>
<thead>
<tr>
<th>Tailings Type</th>
<th>Peak Frictional Strength for Limit-Equilibrium Analyses</th>
<th>Residual Strength for Limit-Equilibrium Analyses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose Sandy Tailings (Q&lt;30)</td>
<td>Not Applicable</td>
<td>$S_u/\sigma_v' = 0.10$</td>
</tr>
<tr>
<td>Loose Silty Tailings (Q&lt;15)</td>
<td>Not Applicable</td>
<td>$S_u/\sigma_v' = 0.06$</td>
</tr>
<tr>
<td>Compact Sandy Tailings (30&lt;Q&lt;70)</td>
<td>If non-brittle, limit-equilibrium with $\phi'=33^\circ$</td>
<td>$S_u/\sigma_v' = 0.15$</td>
</tr>
<tr>
<td>Compact Silty Tailings (15&lt;Q&lt;45)</td>
<td>If non-brittle, limit-equilibrium with $\phi'=28^\circ$</td>
<td>$S_u/\sigma_v' = 0.10$</td>
</tr>
<tr>
<td>Dense Sandy Tailings (Q&gt;70)</td>
<td>Limit Equilibrium with $\phi'=35^\circ$</td>
<td>Not Applicable</td>
</tr>
<tr>
<td>Dense Silty Tailings (Q&gt;45)</td>
<td>Limit Equilibrium with $\phi'=30^\circ$</td>
<td>Not Applicable</td>
</tr>
</tbody>
</table>

For the CANLEX Phase I and III tailings at Syncrude, the BBW material can be assigned the average $(N_1)_{60}$ value of about 12.5. Using the lower bound in Table 9.4, i.e. the 30% to 60% fines content, the resulting estimated $S_u/\sigma_v'$ becomes 0.22. If both the lower bound in Table 9.4 and a lower bound, mean minus one standard deviation, $(N_1)_{60}$ per the CANLEX database of $(N_1)_{60} = 7.8$ are used then an estimate of $S_u/\sigma_v'$ becomes 0.11.

Vaid et al. (1995), as part of the CANLEX project, carried out steady-state testing on a number of Syncrude tailings samples. Table 9.6 presents a summary of their lower-bound results for representative stress path and sample preparation techniques.

### Table 9.6 Summary of CANLEX Laboratory Testing on Syncrude Tailings - Steady State Results

<table>
<thead>
<tr>
<th>LOADING CONDITION</th>
<th>$S_u/\sigma_v'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triaxial Compression (water pluviated)</td>
<td>0.26</td>
</tr>
<tr>
<td>Hollow Cylinder (water pluviated)</td>
<td>0.20</td>
</tr>
<tr>
<td>Simple Shear (water pluviated)</td>
<td>0.16</td>
</tr>
</tbody>
</table>
It is interesting to note how close the recommended values from Table 9.4 are to the CANLEX laboratory data.

### 9.7.3 Practical Implications of Constrained Undrained Strengths

Where there is a potential for a liquefaction trigger, static or dynamic, and the tailings are possibly susceptible to liquefaction, what is the practical implication of this situation? This is obviously a question with a multitude of answers as there are a multitude of situations that can arise. However, a very typical situation, particularly for upstream and even for some centreline constructed facilities, is where pondward tailings are shown to be susceptible to liquefaction yet there is a "shell" of some thickness of non-liquefiable material. The alternative situation is where this "shell" is of insignificant thickness, e.g. for many upstream facilities, and the confinement available is from the potentially liquefiable tailings alone.

To reassess the geometry requirements for the situation where there is a shell, an examination of the geotechnical "need" for this shell of non-liquefiable tailings (or other earthen materials) can be carried out. The undrained destabilizing loading of a liquefied mass of tailings would be approximately equal to:

\[
D = \frac{1}{2} \gamma H^2
\]  

(9.11)

where:

\[
\begin{align*}
\gamma &= \text{total unit weight of liquefied material} \\
H &= \text{deposit height}
\end{align*}
\]
As a first approximation, the shell is ignored and the inherent stability of the potentially liquefiable tailings is estimated. To do this, it is best that the toe resistance, side and backscarp strengths are ignored. Consequently, the resistance supplied to such a destabilizing loading—in terms of undrained strength of the potentially liquefiable tailings (which are assumed to all fully liquefy) can be approximated as:

\[ R = S_u \cdot S \cdot H \]  

(9.12)

where:

- \( S_u \) = undrained strength
- \( S \) = deposit slope (in terms of a H:V ratio)

Combining equations 9.11 and 9.12 by assuming a Factor of Safety of unity (i.e., \( R/D = 1 \)) and eliminating common terms, equation 9.13 is established:

\[ S_u = \frac{\gamma H}{2S} \]  

(9.13)

If the tailings deposit is considered fully saturated, a common condition for most operating facilities, then using the concept of normalized strength \( S_u \) can be assigned as \( (S_u/\sigma'_{vo}) \times H \times (\gamma'/2) \) with the division by two required to give an average strength along the potential plane of shear movement. Using this stress normalization, and substituting the symbol \( C \) for \( S_u/\sigma'_{vo} \), equation 9.13 becomes:
\[ C = \frac{2}{S} \]  

(9.14)

where the ratio of \( \gamma' / \gamma' \) is taken as 2 (saturation assumed complete).

Equation 9.14 therefore presents a simplistic method which, as it ignores several strength benefits, provides a conservative estimate of stable slope \( S \) under full deposit liquefaction and without any structural shell. In other words, the computed value of \( S \) will be the slope where no thickness of a dilatant zone (BAW tailings) is required. As an example, from the CANLEX laboratory data shown in Table 9.6, the slope would therefore be in the range of 12.5H:1V to 7.7H:1V. For the average field value of \( (N_1)_{60} \) for BBW CANLEX Syncrude tailings from Table 9.4, a stable liquefied slope of 9H:1V would be predicted. The lower bound BBW \( (N_1)_{60} \) estimate provides a slope of 18H:1V. To put these slopes in some perspective, Figure 9.31 shows the slopes of some constrained and lesser-constrained slopes following liquefaction events.

In any event, if the existing slope of the tailings is greater than the computed slope without the structural shell, the tailings will likely flow (flowslide development) if there is no shell. If there is a shell, the shell has to provide the additional strength equivalent to the difference between \( D \) and \( R \) when the actual slope of the facility is utilized versus the estimated slope for the fully liquefied tailings.
Figure 9.31 Case Examples of Constrained Slopes Following Liquefaction Induced Failures
The validity of using a stress-normalized value for this type of assessment is by the definition of the assessment; “constrained analyses”. There is the implication that if the tailings do liquefy, they will not be allowed to flow and develop unlimited strain. Under the constraint of their own stress conditions (flat enough slope to preclude flow failure) or a sufficient shell, the tailings are influenced by their initial stress conditions just as residual strength in a clay is under shear loading.

9.7.4 Flowslide Development - Unconstrained Analyses

While there is strong argument for the use of stress normalized undrained strength of tailings under constrained conditions, the Author suggests that such stress normalization is invalid where loss of containment occurs and the tailings can strain in an unlimited fashion (e.g. flowslide). There is no manner for the tailings to acknowledge their stress history upon a total lack of confinement and strain levels of 10’s to 100’s of percent. Under these conditions of non-constraint, what type of evaluation should be used and how does the piezocone provide assistance in these cases?

For tailings deposit evaluations, there are two main methods of liquefaction flowslide inundation prediction carried out in practice. The first of these methods is largely an empirical technique which has been calibrated with a number of case histories. The second is a theoretical approach which considers the tailings to behave as a Bingham fluid upon liquefaction and, by use of a viscosity and yield stress equivalence, inundation is estimated for this dense fluid from an idealized section of the tailings impounding structure.
The empirical method most typically used to calculate tailings runout distances is based on the approach proposed by Lucia et al. (1981) and discussed by Vick (1991). This is a simplified two-dimensional analyses which neglects inertial and viscosity effects. The main components of the method's calculations are shown on Figure 9.32 and are explained below.

The method assumes that the liquefied tailings will flow until a uniform stable slope angle is achieved. The stable slope angle is determined by the height of the tailings runout, $H_r$, and the average undrained shear strength of the tailings runout material. The relationship between the stable slope angle, and a normalized stability factor, $N_o$, is shown on Figure 9.32. $N_o$ is defined as:

$$N_o = \frac{\gamma_t H_r}{S_u}$$

(9.15)

where:

- $\gamma_t$ = total unit weight of tailings;
- $H_r$ = height of tailings runout; and
- $S_u$ = average undrained strength of the tailings.

An angle, $\beta$, represents the angle of the ground surface sloping away from the impoundment. A vertical face of height, $H_c$, is allowed at the terminus of the tailings runout. $H_c$ is calculated as:

$$H_c = \frac{4S_u}{\gamma_t}$$

(9.16)
NOTES:
1. VOLUME CONSERVATION \[ A_1 = A_2 \]
2. TAILINGS RUNOUT DISTANCE
   \[ L_r = \frac{H_1}{2 \tan \alpha} - \frac{H_C}{\tan \alpha} - \frac{H_1}{2 \tan \phi} \]
3. \( \beta = \) ANGLE OF GROUND SURFACE SLOPING AWAY FROM THE IMPOUNDMENT

(a) CALCULATION OF TAILINGS RUNOUT DISTANCE
(b) STABLE SLOPE ANGLE OF TAILINGS RUNOUT

Figure 9.32 Empirical Tailings Runout Estimation Technique (adapted from Lucia et al., 1981)
Conservation of volume dictates that the volume of tailings runout can be no larger than the volume of the original tailings impoundment. This restricts the runout height, $H_r$, and distance, $L_r$, which may occur from impoundments of small volume. In fact, large scale flowslide events from smaller tailings impoundments typically involve between 20% and 40% of the total impoundment volume. However, for large impoundments there is essentially no limit on the volume of tailings available. For the case of a very large impoundment, the runout height, $H_r$, is therefore essentially equal to the initial height of the impoundment, $H_i$ and such an assumption is made for analyses in these situations.

The average undrained strength of the flowslide mass is taken to be the mobilized undrained residual strength of the tailings. As discussed, the mobilized residual strength, which is a minimum assured value, is appropriate for the tailings runout which experiences extensive straining and remolding. Furthermore, a significant portion of the residual strength database is, in fact, backcalculated from flowslide events.

Figure 9.33 shows a reinterpretation of the original Seed database with the addition of several key flowslide runout case histories. Figure 9.33 is adapted from Vick (1991). Figure 9.33 can be utilized by estimating $(N_1)_{60}$ as described in Chapter 8. As will be shown by a case history introduced in this thesis, when the constrained situation has a relatively low stress level (e.g. minimal confinement), or the constraint is only partially present, the constrained normalized approach and that proposed in Figure 9.33 tend to provide essentially the same result.
Figure 9.33  Relationship between \((N_1)_{60-ECS}\) and Undrained Strength Backcalculated from Post-Failure Flowslide Geometries (adapted from Vick, 1991)
A maximum runout distance can be determined using the lower bound of the residual strengths from Figure 9.33 and the method described. This runout estimate tends to represent an extreme case and often does not occur in actual situations due to, among other things, the following:

- the tailings runout will consist of intermixed tailings which range in density and also gradation from loose silty sands to dense, coarse sands;
- the mixing tends to lead to much lower strengths (Byrne, 1998);
- extensive zones of the tailings may be too dense to liquefy and will not flow; and,
- the representative residual shear strength will be a bulk average of all the tailings rather than the lower bound strength of the weakest layer(s).

The second assessment methodology available to predict unconfined behaviour of liquefied tailings is a theoretical approach which treats the tailings as a viscous fluid; i.e., material with a shear strength and a kinematic viscosity. This method was proposed by Jeyapalan et al. (1983) and involves representing the tailings material during the flow event by a Bingham plastic rheological model. For hard mineral tailings, such as present at most base metal, precious metal and oil sands mines, the flow is predicted to be laminar. Using the Bingham plastic model for laminar flow, the method requires evaluating two dimensionless resistance terms, \( R \) and \( S \), defined below.

\[
R = \frac{2 \eta_p}{\gamma H_o} \left( \frac{g}{H_o} \right)^{1/2}
\]

\[
S = \frac{\tau_y}{\gamma H_o}
\]

(9.17)  
(9.18)
\[\eta_p = \text{plastic viscosity}\]
\[g = \text{gravitational acceleration}\]
\[\gamma = \text{tailings total unit weight}\]
\[\tau_y = \text{yield shear strength}\]
\[H_o = \text{tailings impoundment height}\]

The method assumes all of the tailings liquefy to the same low strength, start at a "box-like" structure and then flow to an inundation distance within a calculable freezing time. The dimensionless parameters R and S are used to determine dimensionless inundation and freezing time values, \(x_f\) and \(t_f\) respectively, as per nomographs given in Jeyapalan et al. (1983). The undrained tailings strength from Figure 9.33 can be substituted for \(\tau_y\) in Equation 9.18.

### 9.7.5 Sullivan Tailings Flowslide - Case Example of Static Liquefaction

#### 9.7.5.1 Background

The Cominco Metals/Sullivan Mine located near Kimberley, British Columbia is an underground lead/zinc mine which is currently processing an average of about 8000 tons per day of feed. The mine was established in 1905 and is scheduled for closure in 2001. Active closure activities have been taking place at the mine throughout the 1990s.

From the start of operations, all of the mine tailings have been hydraulically transported to an area southeast of the concentrator. A series of separate areas have been developed over the years by constructing a system of containment dykes as necessary. The only presently active tailings pond is the Active Iron Pond (AIP) which is surrounded by several inactive tailings ponds. A general layout of the tailings impoundment area is shown on Figure 9.34.
Figure 9.34  Sullivan Mine Tailings Area - Site Plan
The AIP is formed by about 1500 m of earthfill dykes which presently have a maximum height of approximately 21 m. The starter dykes for this pond were constructed in 1975 and were incrementally raised using the upstream method of construction. The exterior shell of mechanically placed and compacted tailings was progressively "stepped" upstream directly onto the previously spigotted tailings beach. Portions of the dykes were founded on the native ground, while other portions were constructed over previously deposited tailings. Design and regular inspection of the AIP facility to 1991 was carried out by consulting engineering firms.

On August 23, 1991, about 2:00 P.M., the southeastern portion of the dyke between approximately stations 29+00 and 39+00 suddenly moved (slumped) during construction of a 2.4 m incremental raise of the dyke. The location of the slumped section is shown on Figure 9.34. Figures 9.35 and 9.36 show aerial and ground views, respectively, of the movement two days after the event. The event occurred relatively quickly, just as the final lift of the dyke raise was being placed. Construction at the time was such that loaded scrapers from the borrow pit located north of the site would travel along the dyke crest to spread their loads. Borrow for dyke raises up to and including 1991 was from coarse siliceous tailings. Empty scrapers would return to the borrow pit by running down a construction road on the dyke slope and onto a lower access road at the toe of the iron dyke. It was reported by eye witnesses that an empty scraper had punched through the lower return road, immediately preceding the event.
Figure 9.36
Sullivan Tailings Liquefaction Slump Event - Ground View
The area of the slump event was about 300 m long and 12 m high, and involved up to an estimated 75,000 m$^3$ of materials. The toe area was observed to have been displaced about 15 m to 45 m downstream of its original position although there was clear evidence of disturbance for a distance of up to about 100 m downstream of the original toe. The final slopes of the slumped mass averaged about 10H:1V to 15H:1V. Visual observations indicated that the compacted dyke sections and roadways basically remained intact, but they did experience some brittle rupturing. Numerous sand boils were observed issuing from the ground surface just after the failure and for several hours later. In addition, the surface of the toe area which was mainly dry immediately after the event, became wetter over the next 24 hours as water seeped from the movement zone. Fortunately, there were no casualties resulting from the event. The pooled water in the AJP was located sufficiently far from the failed section to remain unaffected and, as the slide materials were adequately contained within the No. 2 Siliceous Pond, there were no spills to the environment. Monitoring of survey points that were immediately installed within the area of the event showed little or no additional movements following the initial event.

Based upon a review of key visual observations, data records and the nature of the movement, it was concluded that the tailings within the foundation of the AJP dyke had liquefied. It is probable that triggering of liquefaction was contributed to by the additional loadings on the crest due to fill placement during the dyke raising, and/or by the dynamic loadings imposed near the toe due to construction equipment traffic. However, there is stronger evidence indicating that the lower portions of the dyke slope may already have been in a state of incipient failure at the time. It is likely that the liquefaction event might have eventually occurred even if construction of the latest dyke increment had not taken place.
9.7.5.2 In-Situ State

Following the slump event, a total of 26 sampled boreholes and 42 piezocone soundings were carried out along the AJP dyke alignment. Of these, 11 boreholes and 22 piezocone soundings were placed within the immediate vicinity of the slumped zone. The key findings from the in-situ investigations included:

- In general, the entire dyke along the area of the slump was founded directly on spigotted iron tailings. The total thickness of the basal tailings ranged from about 15 m beneath the 1991 dyke extension to about 8 m beneath the 1979 dyke. Relatively dense and competent glacial tills and/or sands and gravels exist below the tailings. Typical cross-sections showing the general distribution of the units within the liquefaction event area and a reconstructed section are presented on Figure 9.37.

- The foundation tailings were typically fine-grained, having about 50% or greater passing the No. 200 particle sieve size (74 μm), and classified as a non-plastic "silt" and/or "silty fine sand". Specific gravities of the iron tailings and the silica tailings were determined to be in the order of about 4.2 and 3.3, respectively. These values were used in computing accurate stress states for evaluating the event.

- Very loose and weak zones were evident within the tailings that formed the foundations for the AJP dyke. Energy-calibrated SPT and the piezocone work was used to provide \( (N_i)_{60} \) values. Although there were distinct zones which were relatively dense with \( (N_i)_{60} \) values in the order of 35 to greater than 40 blows/ft (some of the higher values correspond to the compacted dyke extensions), a large portion of the tailings deposit was loose and weak, with \( (N_i)_{60} \) values as low as 2 to 5 blows/ft. The peaks of higher \( (N_i)_{60} \) values at depth are likely indicative of tailings surfaces that were exposed and oxidized between spigotting cycles.

- As noted by some example profiles earlier in this Chapter, the state parameter of the looser zones, determined mainly from piezocone data, but with some laboratory calibration, ranged from \( \psi = +0.01 \) to \( \psi = +0.12 \). The upper bound values are likely as contractant as spigotted tailings can readily achieve and appear looser than can be achieved in most laboratory environments.

- Also as noted earlier in this Chapter, the seismic piezocone shear wave data showed wave velocities more typical of neutrally contractant if not dilatant soils. The oxidation processes may have "stiffened" the tailings to small strain shear (e.g. from shear wave testing) but these bonds had no effect in either limiting or restricting brittle behaviour upon appreciable shear strain levels.
The south-east corner of the AIP dyke was historically a very wet area, with the first report of potential seepage problems coming in 1980, shortly after commencement of spigotting operations along the 1979 dyke extension. Over the years, numerous piezometers were installed in the area to monitor groundwater levels. Time history plots of the readings from all of the piezometers exhibited a general rise in levels from 1980 as would be expected with the rising pond levels. However, some of the trends during the 1991 construction were in excess of this general rise and readily indicative of potential concern. For instance, it is interesting to note that the last available readings prior to the dyke liquefaction event (July 16, 1991 readings) indicated that the groundwater level near the toe of the Iron dyke was well above the ground surface.

9.7.5.3 Assessment of Dyke Failure

Available evidence from visual observations of the slump area, as noted above, suggests that the tailings within the foundation beneath the dyke had liquefied. The triggering of liquefaction was precipitated by the slump event.

Over the course of a review of the event, various methods of analyses were used to investigate the probable cause and mechanism. The review showed that, while local liquefaction of the near surface tailings by scraper traffic was a potential contributory trigger, the extent of the liquefaction event was likely the result of static instability near the toe of the dyke. More details on pre- and post-slide stability analyses are presented in Davies et al. (1998).
A key element of the remediation design involved the selection of the appropriate undrained or residual strength for the essentially cohesionless tailings. Despite significant advances in the field of earthquake/liquefaction engineering over the last decade, there were uncertainties associated with this task, with the final selection of the design strength requiring considerable judgement.

The slump event has provided a unique opportunity to back-analyze the field residual strength values for the liquefied tailings. The accuracy of the strength obtained from this method, however, depends on the key assumptions that are used in the back-analysis, including the extent and geometry of the failure surface, the extent of the liquefied zone, and the strength parameters for the nonliquefied portions of the dyke. There was insufficient evidence from the field investigations or from the field observations to accurately determine the first two items. Nevertheless, reasonable assumptions were made whereby the event was determined to have occurred in two main sequences, and that the sliding surface appeared to have been relatively shallow.

The computed residual strength of the liquefied tailings to achieve a factor of safety near unity for the selected geometry was about 10 kPa (~200 psf). It is interesting to note that the back-calculated residual strength of 10 kPa (~200 psf) generally lies within the bound of Seed's curves (Seed and Harder, 1990) or per Figure 9.33.

As a crude check of the residual strength for the iron tailings at the Sullivan Mine, the method per Lucia et al. (1981) was used to compute the applicable residual strength value for the observed "runout" distances in the failure area. The result yielded a residual strength value of about 9.5 kPa (190 psf) to 15 kPa (300 psf), depending on various assumptions that were made. Although crude,
the results of this simplified analysis also suggest that the back-calculated residual strength of 10 kPa (200 psf) from limit equilibrium considerations is reasonable.

Finally, the stress-normalization residual strength estimation technique per constrained tailings proposed earlier was used based on the \((N_1)_{60}\) data. Using the earlier proposed relationship for 30% to 60% fines, a range from \(S_u / \sigma'_{vo} = 0.06\) to \(S_u / \sigma'_{vo} = 0.09\) would be predicted. As the effective vertical stress level at mid-height of the toe area was about 125 kPa, the estimated undrained strength is 7.5 kPa (150 psf) to 11.25 kPa (225 psf).

Overall, it is interesting to note that both the unconstrained and the constrained approaches provided good agreement with the observed behaviour and with one another. As noted earlier, at least a portion of this agreement is due to the low stress level present for this case history event.

9.7.5.4 Summary of Case History

The 1991 liquefaction slump of mine tailings within the upstream constructed Iron Tailings Dyke at the Sullivan Mine involved approximately 75 000 m\(^3\) of materials. No release of materials to the environment (e.g., offsite) occurred as a result of the event. Immediate measures were taken to maintain this non-release situation with full reconstruction of the dyke being carried out in 1992. The event caused tailings deposition to be halted for less than one week. Another fortunate aspect of the failure was that in spite of manned-activity during the event, no injuries occurred.
From a geotechnical perspective, the key conclusions from this case history include:

- the pre-failure dyke slope was in the range of 2.5H:1V to 3H:1V, which is close to the contractive and/or extensive collapse surface for fine-grained cohesionless tailings;
- excess pore pressure rise immediately prior to the event clearly occurred;
- back-calculated strengths for the liquefied tailings yielded a $S_u / \sigma'_{yo}$ value of about 0.08;
- in-situ shear wave velocity measurements were not consistent with the brittle behaviour when published liquefaction susceptibility criterion are applied; and
- forensic limit-equilibrium analyses, using pre-event available data, showed the tenuous nature of the dyke when correct effective stresses were used in the analyses.

9.7.6 Moshikoshi Tailings Flowslide - Case Example of Seismic Liquefaction

Failure of tailings impoundments from seismic loading can result in a breach of the embankment and release of tailings in the form of a flow failure. A typical case history illustrating this failure mode is the Moshikoshi tailings dam in Japan that failed in 1978 during a M7.0 earthquake. The general plan of the Moshikoshi tailings facility, dam sections and schematic nature of the failures is shown on Figure 9.38. Okusa and Anma (1980), Okusa et al (1980), and others report details of the facility and the failure. Dyke No. 1 failed directly after the main shock of the M7.0 earthquake that occurred 37 km from the site. Dyke No. 2 failed 5 hours later during an M5.8 aftershock occurring 5 km from the site. Failure of Dyke No. 1 resulted in a flow of about 80,000 m$^3$ of tailings 8 km down a river valley causing one death and serious environmental impacts. Dyke No. 2 resulted in flow failure of 3,000 m$^3$ which only flowed about 150 m from
Figure 9.38  Schematic Details of the 1978 Moshikoshi Tailings Flowslide (adapted from Okusa and Anma, 1980)
the dam due to apparent drier condition of these later tailings and the lack of a transport mechanism (e.g., creek or similar) at the toe area.

The Moshikoshi tailings consisted of tailings from crushed quartz veins hydraulically transported to the tailings facility at 35% solids density and discharged into the impoundment from a pipe at each dyke and a pipe between Dykes 1 and 2, resulting in an average rate of deposition of 2.2 m/year. The tailings are reported to be fairly uniform around the impoundment with a slight decrease in gradation from the edges of the dyke to the centre of the ponds. The total area of the pond was about 30,000 m².

The tailings consisted of inter-layered sandy silt and silt. Importantly, the observed water content of the tailings was above the liquid limit of the materials (liquidity index > 1) which indicates the material was loose and flowable when subject to disturbance. The back-calculated undrained strength of the liquefied material at depth agreed well with the lower bound undrained shear strength of 18 kPa to 21 kPa interpreted from Dutch cone penetration tests conducted after the earthquake event.

Ishihara et al. (1989) note that both of the slides appear to have been initiated by liquefaction at about a depth of 6 metres. Mechanical cone data available indicates that \( q_c \) at that depth was in the range of 2 to 4 bar. Ishihara et al. (1989) suggested that a value of \( N_c \) for the Moshikoshi site \( (N_c = q_c/S_u) \) would be in the range of 10 to 20; not much different than the value of 10 indicated in Chapter 8 for \( N_{KT} \). At the prevailing stress level of the failure initiation, \( N_{KT} \) would be roughly \( 2/3 \) to \( 3/4 \) of \( N_c \).
Assuming essentially half-depth saturated conditions pre-failure, and using the residual strength of between 16 and 18 kPa per Ishihara et al. (1989), a value of $S_u/\sigma_{vo}$ for the case history is approximately 0.18. A Q value for the material, assuming the intermediate value of $q_c$ of 3 bar would be 2. Clearly, based upon Table 9.5, the strength would have been quite conservatively under-predicted. Moreover, with the fines content equal to approximately 50% (Ishihara, 1984) an $I_c$ can be estimated at roughly 2.5. With a value of $I_c$ about 2.5, the equivalent $N_{60}$ value would be about 5 and the $(N_1)_{60}$ value not being much different. Again, from the proposed methods noted earlier in this Chapter, a value of $S_u/\sigma_{vo}$ approaching 0.18 would not have been predicted. Using the proposed method summarized by Table 9.4 for 50% fines, and value of $S_u/\sigma_{vo}$ of roughly 0.06 would be predicted.

The Moshikoshi case history provides an indication that the procedures proposed in this thesis may be conservative in some cases.

### 9.7.7 Cyclic Piezocone

As noted in Chapter 4, initial efforts to determine the potential for direct measurement of a high-strain undrained strength with the friction sleeve of the piezocone was carried out as part of this research. During the research three different aspects were reviewed:

- assessing standard $f_s$ data and comparing the values to "expected" strength values;
- assessing $f_s$ data from "faster" penetration testing where the hydraulic pushing rate was increased in increments to a maximum of about 10 cm/s to see if (more) undrained conditions in silts and sands could be obtained; and
- developing and assessing "cyclic" piezocone testing with both modified hydraulic and data acquisition conditions.
As noted in Chapter 4, $f_s$ measurements are partially to wholly steel-soil measurements of frictional resistance versus a soil-soil measure. For normally consolidated conditions in silty sand to sand, $f_s$ can be expected to be linearly related to the effective overburden stress as approximately:

$$f_s = 0.15\sigma'_v$$  \hspace{1cm} (9.19)

However, as noted in Chapter 4, the effective stress around the cone is not trivially derived. The pore pressure within the soil mass adjacent to the friction sleeve is neither hydrostatic nor equivalent to either $U_2$ or $U_3$ along its entire length. This condition is not a large issue in fully-drained penetration (e.g., coarse sands), but for the majority of mine tailings, partially undrained conditions exist during penetration and complex shear-induced pore pressures are developed.

For the Kidd 2 and Massey South CANLEX sites in the Lower Mainland, the silty sand and sand are essentially normally consolidated. A combination of normal (2 cm/s) and variable penetration rate testing was carried out to investigate the relationship between the sleeve friction, overburden stress and the estimated value of undrained strength for the in-situ materials.

Figure 9.39 shows some of the typical data from the Kidd 2 site. From the CANLEX project (Robertson and Wride, 1997), laboratory samples from Kidd 2 in triaxial extension showed $S_u/\sigma_{vo}' = 0.09$. Testing in triaxial compression and simple shear failed to produce sufficient strain softening to determine such a strength ratio.
Figure 9.39, as did all of the similar data, showed a reduction in the \( f_s/\sigma_v' \) ratio as penetration resistance increased. However, truly undrained conditions, as assessed by the measurement of pore pressure at both U2 and U3, did not develop. It is interesting to note that the lower bound of the data does indeed approach the predicted ratio of 0.15 per Equation 9.19.

To attempt to create a testing scenario closer to an undrained soil-soil shear loading, two modifications to the testing methodology were made:

1. a ridged friction sleeve, as shown schematically in Figure 4.18, was used to assist in maintaining a soil-soil frictional response, even at high strains; and
2. the hydraulic pushing arrangement was modified to allow faster, and cyclic, penetration whereby undrained conditions in at least some largely cohesionless tailings could be achieved. Data acquisition was achieved by increasing the rate of measurement under the pore pressure dissipation mode of GEODAS.

Due to the hydraulic servo-controller in the ISTG vehicle, the second adjustment was only partially successful. Hydraulic switching and build up of entrained air in the system only allowed several (up to about 15) significant loading cycles of roughly 20 cm in amplitude prior to loss of hydraulic pressure and hence control. The cyclic testing was not an ideal test for the available systems within the ISTG vehicle.

Notwithstanding the hydraulic system challenges, there were indications in the testing that the initial research effort provided some interesting, if not encouraging, results. Even at normal pushing rates, the ribbed friction sleeve did appear to provide somewhat higher \( f_s \) values in all materials tested including the Sullivan Mine tailings (the only tailings tested with these
Figure 9.39 Variable Rate Piezocone Testing - Effect on $f_a/\sigma'_{v}$
modifications). The approximate increase was in the order of 20% to 30%, which is roughly the
difference in the tangent of the frictional resistance change from largely steel-soil to largely soil-
soil. It is therefore possible, if true soil-soil testing was or can be achieved, that a direct
measurement of $\Phi'$ in-situ may eventually be possible with the piezocone, although accurately
determining the effective stress normal to the friction sleeve is not currently possible.

The cyclic testing was applied at both one minesite and at the Kidd 2 calibration site. The cyclic
testing was not very successful at the Sullivan Mine in an area adjacent to the 1991-liquefaction
slump. Although pore pressures did increase, the values of $f_s/\sigma_v'$ were highly variable with a
range from 0.02 to 0.19 based upon ten tests. At the same time, from the back-calculations
summarized in Davies et al. (1998), this relatively wide range of values does bracket the likely
value of $S_u/\sigma_v' = 0.08$ in these tailings.

At the Kidd 2 site, Figures 9.40 and 9.41 show some of the typical data obtained from the cyclic
piezocone testing. At 5.8 m, the ratio of $f_s/\sigma_v'$ became asymptotic to 0.08 whereas at 10.8 m, the
ratio was about half that value at 0.04. Both values were lower than the $S_u/\sigma_v'$ ratio of 0.09
reported by Robertson and Wride (1997). In both cases, it is possible that the modified sleeve
still did not result in true soil-soil interaction. It is also difficult to know whether truly undrained
conditions were present. In addition, the stress path of the sleeve friction shear is not
commensurate with triaxial extension.
Pore Pressure ($U_2$) and Skin Friction ($f_s$) vs. Time (sec)

Figure 9.40  Cyclic Piezocone Testing - Kidd No. 2 at 5.8 m
Pore Pressure ($U_2$) and Skin Friction ($f_s$) vs. Time (sec)

10.8m

Figure 9.41 Cyclic Piezocone Testing - Kidd No. 2 at 10.8 m
Overall, the research only briefly examined the concept of using the cyclic piezocone. The results from this examination, summarized above, do not wholly endorse or condemn the concept. More research is required as to whether values of $f_g/\sigma_v'$ values, either from cyclic or standard piezocone testing at variable rates, can be related with $S_u/\sigma_{vo}'$. From the preliminary results, which are far from conclusive, the values $f_g/\sigma_v'$ were at least within reasonable range of corresponding $S_u/\sigma_{vo}'$ values.

9.7.8 Summary

The piezocone technology currently available to assess mine tailings appears well suited to assist in the estimation of the post-liquefaction undrained strength of those materials. A number of different existing and proposed methods were presented in this Chapter. The user of any of these methods must be aware of what any of the values represent; estimates for careful review with appropriate engineering judgment. The distinction between constrained and unconstrained analyses was made and such a distinction is essential prior to choosing any estimation method.

Tables 9.4 and 9.5 offer algorithms that can be readily automated and added to any piezocone assessment software for screening purposes. A more useful estimation method may be one consistent with the Integrated Piezocone Approach used throughout this thesis for state screening and the assessment of $I_c$. Consequently, Figure 9.42 is proposed as being a useful screening tool consistent with the state parameter assessment charts used elsewhere in this Thesis. The contoured values of $S_u/\sigma_{vo}'$ are determined from the equations developed earlier in this Thesis which related Q-F pairs to $\psi$ and then the relationship earlier in this Chapter relating $\psi$ to $S_u/\sigma_{vo}'$ (equations 9.10 and 7.9 with assumptions of $K_o = 0.5$, $M = 1.2$). Adjustments to Figure 9.42, or
the algorithm used to create the contours of equal $S_u/\sigma_{vo}^*$, can be made by using site specific values of $K_o$ and $M$. Equation 9.20 presents the relationship used to develop the contours of $S_u/\sigma_{vo}^*$ on Figure 9.42.

$$
\frac{S_u}{\sigma_{vo}^*} = (0.5)(10)^{\left[ \frac{\left[ Q(1 - B_o) \right]}{(1.33F - 11.9)} \frac{10}{F} \right]}^{-1}
$$

(9.20)
Figure 9.42 Proposed Screening Level Estimate of $S_u/\sigma_{vo}$ for Mine Tailings from Integrated Piezocone Approach

NOTE: RATIOS OF FIGURE ARE $S_u/\sigma_{vo}$ PREDICTED FROM Q/F RELATIONSHIP WITH $\psi$. 

Q(1-Bq)

F (%)
10. HYDROGEOLOGICAL PARAMETERS

10.1 Modeling Requirements

Mine tailings facilities are complex hydrogeological systems. Due to the typical nature of their construction, true three-dimensional anisotropy in hydraulic parameters is more the rule than the exception. For most saturated and unsaturated flow conditions in mine tailings, the movement of pore fluids is laminar and can be described quite adequately using Darcian flow rules. Using the convention of conservation of fluid mass, the universal flow equation for porous media where conditions are not isotropic is:

\[
\frac{\partial}{\partial x}(K_x \frac{\partial h}{\partial x}) + \frac{\partial}{\partial y}(K_y \frac{\partial h}{\partial y}) + \frac{\partial}{\partial z}(K_z \frac{\partial h}{\partial z}) = \text{RHS} \tag{10.1}
\]

RHS = “right hand side” of Equation 10.1 and can be zero for steady state conditions or non-zero when there is a transient condition (e.g. \( \frac{\partial h}{\partial t} \neq 0 \)).

For this thesis, the x and y plane is taken to be the horizontal and the z-direction coincident with depth as measured normal to level ground surface.

To describe any tailings facility for its hydrogeological character, the following requirements are indicated from the governing flow equation:

- a discretization of the facility into “like” units (homogeneous units);
- estimation of the hydraulic conductivity matrix for each unit;
- estimation of the gradients/heads within each of the units; and
- assessment of any transient conditions.
Piezocone data can be used to determine like units through the material index, $I_c$. Such concepts are described earlier in this thesis. Typically, there is not any need to further define units for hydrogeological purposes other than what would be done for basic stratigraphic logging or geomechanical unit discretization. On the other hand, estimating the hydraulic conductivities and in-situ gradients (though each can be non-uniquely obtained from the other by using Equation 10.1) are fundamental hydrogeological requirements and the focus of the portion of research into piezocone technology capability in mine tailings.

10.2 Water Table and Gradients

Using the piezocone for assessing the location of the water table (zero pressure head) and vertical to sub-vertical in-situ gradients is relatively straightforward in many soil deposits. However in mine tailings, particularly tailings embankments with pervious foundations, the task is more difficult. As per the example data provided later in this section of the thesis, it is often very difficult with piezocone data alone to determine the location of the zero pressure condition and, consequently, not possible to determine in-situ gradients. However, as will be demonstrated, using the resistivity piezocone can potentially eliminate this problem.

Figure 10.1 shows very simplified schematics of the two end-members for seepage conditions within typical tailings impoundment embankments. On Figure 10.1a, the seepage is lateral and essentially entirely reports on the downstream face of the tailings embankment. A standpipe piezometer placed at the embankment crest would be coincidental with the water table that, in this case, would also be a phreatic surface. However, in Figure 10.1b, there is a substantial
Figure 10.1 Typical Seepage Patterns in Mine Tailings Impoundments
downward gradient as the pervious foundation provides the tailings with a desirable drainage path in addition to the free-face of the embankment. In this latter case, a standpipe piezometer placed at the embankment crest would not measure the correct location of the water table.

To better illustrate the phenomenon of the pervious foundation, Figure 10.2 shows some typical flowlines and equipotential lines for a hypothetical tailings embankment on pervious foundations. Besides illustrating the problems the standard piezocone will have in determining the water table, Figure 10.2 also shows the inability for single installation standpipe piezometers to locate the water table accurately where any appreciable gradient is present.

The problems that standpipe piezometers have under gradients also exist for piezocones if the downward gradient is high enough. The pore pressure transducer within the piezocone measures the total water pressure at a given point and, if the gradient is high enough, this pressure becomes close to zero even in fully saturated conditions. Pore pressure dissipations can be used to assess the actual equilibrium pressure, as noted in Chapter 4, but if the gradient is high enough (e.g. greater than about 0.7), dissipation data may also be difficult to interpret.

An example of a piezocone sounding from the Endako Mine's Tailings Pond No. 1 is shown on Figure 10.3. Note that the pore pressure profile, U2 in this case, is very non-descript. The pore pressure dissipations from ENDK9510 were manageable but in each case, there was extremely sluggish response in the transducer and the equilibrium pressure was consistently near zero (i.e. implying a downward gradient approaching unity). ENDK9510 was carried out at the crest of
Figure 10.2  Schematic Example of Effect of Seepage Path on Standpipe Piezometer Reading
Figure 10.3  Resistivity Piezocone Sounding ENDK9510 - Endako Tailings Pond No. 1
the North Dam of Tailings Pond No. 1. In that area, coarse fluvial sediments underlie the impoundment. The combination of the slope of the dam and the pervious foundations has created a very high downward gradient in this area.

One of the important reasons to know the true location of the water table comes when assessing geotechnical condition of a tailings dam. For example, if a liquefaction assessment is carried out, any materials below the water table need to be assessed for their susceptibility to liquefaction. Saturation is the only important criteria if the material is at an in-situ state looser than is required to withstand the transient load. Moreover, if there is a downward gradient, the stress levels increase and this increased stress level, given equal in-situ density, leads to defining more potentially contractant materials upon analyses.

The addition of the resistivity module to the piezocone greatly enhances the ability to compute gradients. For example, with the piezocone data alone, it would not have been possible to assess the gradients accurately with ENDK9510 as locating the water table was so difficult. Even with nested standpipe piezometers, the estimate of gradients will be incorrect unless the correct water table elevation is utilized. Note the response of the resistivity channels on Figure 10.3. The substantive difference in measured bulk resistance between saturated and unsaturated materials allows the water table in any sounding to be located with confidence. With the knowledge of water table location, the line of zero pressure can be drawn and accurate gradients determined.
Figure 10.4 shows an example of some of the sections developed for Pond No. 1 at the Endako Mine. Note the difference between the resistivity deduced location of the water table and that indicated by the piezometers present.

Gradient analyses were carried out for essentially every piezocone sounding in the tailings database. Figures 10.5 to 10.7 show typical results from such gradient calculations. These types of figures are developed once the location of the water table is determined from the resistivity information and the pore pressure information is assessed with depth from the dissipation tests. Note on Figures 10.5 and 10.6 that the method also allows the determination of dual gradients that are common where changes in hydraulic properties with depth occur (e.g. due to changes in tailings character over time).

10.3 Hydraulic Conductivity

10.3.1 General

With the location of the water table determined and the in-situ gradients assessed, the next information required for hydrogeological characterization is the in-situ hydraulic conductivity(s). Hydraulic conductivities likely range more than any other parameter in the geoenvironmental field. The typical tailings properties noted in Table 3.1 showed up to 6 orders of magnitude difference in in-situ hydraulic conductivity of mine tailings. When the estimation of travel time of, for example, an advective plume of tailings influenced groundwater is directly proportional to the hydraulic conductivity, it is seldom the case when it is acceptable to potential parties
Figure 10.5  Gradient Analyses - ENDK9603 - Endako Tailings Pond No. 1
Figure 10.6  Gradient Analyses - ENDK9605 - Endako Tailings Pond No. 1
Figure 10.7  Gradient Analyses - ENDK9610 - Endako Tailings Pond No. 1
impacted by the plume that such an estimate can vary by even an order of magnitude. Whether there is a potential problem in 1, 10, 100 or 1,000 years is important to know. Reactive measures such as detection wells will be required in any case but having the best possible estimate of travel times is of prime importance to the designers and owners/operators of tailings facilities.

10.3.2 Saturated Values

Typical values for saturated tailings hydraulic conductivities can be found in literature (e.g. Vick, 1990). Table 10.1 provides some typical values used in this research to check the reasonableness of the estimated values from piezocone data. Also included in Table 10.1 are the approximate equivalent materials from the tailings database available to this research.

### Table 10.1 Average Value of Bulk Hydraulic Conductivity for Typical Mine Tailings
(adapted from Vick, 1990)

<table>
<thead>
<tr>
<th>Processed Material</th>
<th>Tailings Database &quot;Equivalent&quot;</th>
<th>Lower Bound K (m/s)</th>
<th>Upper Bound K (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oil Sand Tailings</td>
<td>CANLEX Phase I and III (Syncrude)</td>
<td>$5 \times 10^{-6}$</td>
<td>$1 \times 10^{-4}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e = 0.65)</td>
<td>(e = 0.8)</td>
</tr>
<tr>
<td>Cycloned Copper Sand</td>
<td>Gibraltar Dyke</td>
<td>$1 \times 10^{-5}$</td>
<td>$8 \times 10^{-5}$</td>
</tr>
<tr>
<td>Tailings</td>
<td></td>
<td>(e = 0.75)</td>
<td>(e = 1.05)</td>
</tr>
<tr>
<td>Copper Sand Tailings</td>
<td>Gibraltar Pond</td>
<td>$4 \times 10^{-7}$</td>
<td>$1 \times 10^{-6}$</td>
</tr>
<tr>
<td>($\leq 35%$ fines)</td>
<td></td>
<td>(e = 0.62)</td>
<td>(e = 0.95)</td>
</tr>
<tr>
<td>Lead-Zinc Fine Tailings</td>
<td>Sullivan Iron Ponds</td>
<td>$8 \times 10^{-9}$</td>
<td>$5 \times 10^{-8}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e = 0.62)</td>
<td>(e = 1.05)</td>
</tr>
<tr>
<td>Copper Fine Tailings</td>
<td>Kennecott Pond</td>
<td>$5 \times 10^{-9}$</td>
<td>$2 \times 10^{-8}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e = 0.62)</td>
<td>(e = 1.22)</td>
</tr>
<tr>
<td>Molybdenum Fine Tailings</td>
<td>Endako Pond</td>
<td>$1 \times 10^{-8}$</td>
<td>$3 \times 10^{-7}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e = 0.68)</td>
<td>(e = 1.35)</td>
</tr>
<tr>
<td>Molybdenum Sand Tailings</td>
<td>Endako Dyke</td>
<td>$1 \times 10^{-5}$</td>
<td>$4 \times 10^{-5}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e = 0.62)</td>
<td>(e = 0.8)</td>
</tr>
</tbody>
</table>

Note: (e = 0.8) values in Table are void ratio values noted by Vick (1990) for each material and range
Following are several methods available with piezocone, and related in-situ technology, for estimating the saturated hydraulic conductivity of mine tailings. Examples from the research database are used to show likely range of applicability for each method described.

10.3.3 Pore Pressure Dissipation Methods

During penetration, pore pressures above or below hydrostatic values (e.g. dependent upon the propensity for a material to contract or dilate under loading) are termed excess pore pressures. The excess pore pressure ($A_u$) measured during penetration is a useful indication of soil type and provides an excellent means of detecting details in soil stratigraphy (Davies and Campanella, 1995). For clean sandy soils excess pore pressure dissipates almost immediately, while for more fine grained clayey and or silty materials of relatively low permeability, significant excess pore pressures can be generated. Excess pore pressures can be either positive or negative depending upon measurement location and soil behaviour. Normally consolidated silts and clays tend to develop large excess pore pressures, whereas overconsolidated silts and clays tend to develop smaller positive or even negative pore pressures. As noted in Section 6, the inclusion of excess pore pressure measurements into the identification of soil type improves the interpretation in finer soils while leaving the interpretation in sands unchanged.

Excess pore pressure measurements also provide valuable insight into the hydraulic parameters of the porous media. When penetration ceases, e.g., after a 1-metre rod push, any excess pore pressures generated during cone penetration will start to dissipate. The amount of pore pressure generated, as noted above, is dependent upon soil type and pore measurement location (i.e. $U_1$, $U_2$ or $U_3$). The rate of dissipation is dependent upon the coefficient of consolidation which, in
turn, is dependent upon the compressibility and hydraulic conductivity of the soil. Dissipation profiles for specific depths were acquired throughout the field programs. All dissipations have the initial negative/positive excess pressures trend towards an equilibrium value (i.e. zero excess pore pressure) after some period of time.

Due to the geometry of the piezocone, radial cavity expansion theories are directly applicable to the data in determining the time rate of consolidation from classical consolidation theory (e.g. Lunne et al., 1997). The shape of the dissipation curves in finer-grained soils are typically amenable to the cavity expansion theory. Estimates of the coefficient of consolidation, $c_{v,h}$ (where $v$ and $h$ are vertical and horizontal coefficients, respectively), are obtained by measuring the rate of dissipation of the excess pore pressure and then applying closed-form linear cavity expansion solutions. Knowing $c_{v,h}$, the hydraulic conductivity ($K_{v,h}$) can then be estimated using:

$$K_{v,h} = c_{v,h} \times m_{v,h} \times \gamma_w$$

(10.3)

where $m_{v,h}$ is the coefficient of volume compressibility in either the vertical or horizontal plane, which can be estimated from piezocone data, and $\gamma_w$ is the unit weight of water. In addition, pressure head distribution within the saturated zone can be estimated based on the equilibrium pore pressure data for all soil types.

The dissipation method has been shown to provide good estimates of hydraulic conductivity in finer-grained soils; e.g. silts and clays. As such, there are ample literature for the "type-curves" needed for both radial and cavity expansion solutions for hydraulic conductivity values in the
range of dissipation time to, say, 50% ($t_{50}$). However, there was no such equivalent for sand materials and the range of conductivities many coarser tailings would possess. As part of this research, values of U-T were established for coarser materials (Campanella et al., 1995). However, it is not considered reasonable to use undrained cavity expansion solutions for drained soils. Alternative theory, not investigated during this research, is required for these coarser soils. An additional requirement for the coarser materials, which was added during the research, was much faster data acquisition. GEODAS was altered to allow enough information to be obtained during the rapid dissipation of sandier materials where $t_{50}$ values could be just fractions of a second versus several seconds to even several minutes or more for finer-grained materials.

The hydraulic conductivity estimates in the finer-grained tailings using pore pressure dissipations was quite satisfactory where corroborating information was available. Both in-situ and laboratory values were available for the INCO and Sullivan tailings and the piezocone dissipations at these sites provided reasonable (e.g. within an order of magnitude) values. Table 10.2 shows some of the typical information from the research program. Both spherical and cylindrical cavity expansion solutions were used and, per Table 10.2, at most mine sites the cylindrical solutions appeared to provide the most consistent estimates when compared to other methods.
Table 10.2  Example of Piezocone Dissipation Data from Tailings Database - Sullivan Old Iron Pond

<table>
<thead>
<tr>
<th>Sounding</th>
<th>Depth (m)</th>
<th>$t_{50}$ (s)</th>
<th>$T_{s}$ spherical</th>
<th>$T_{c}$ cylindrical</th>
<th>$c_s$ spherical (m²/s)</th>
<th>$c_h$ cylindrical (m²/s)</th>
<th>$q_c$ (kN/m²)</th>
<th>$m_h$</th>
<th>$K_h$ spherical (cm/s)</th>
<th>$K_h$ cylindrical (cm/s)</th>
<th>Calibration Well (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>sv7-9510</td>
<td>8.93</td>
<td>4.84</td>
<td>0.9</td>
<td>5</td>
<td>5.92E-05</td>
<td>3.29E-04</td>
<td>4533</td>
<td>5.52E-05</td>
<td>3.20E-06</td>
<td>1.78E-05</td>
<td>3E-05</td>
</tr>
<tr>
<td>sv9-9514</td>
<td>5.40</td>
<td>3.24</td>
<td>0.9</td>
<td>5</td>
<td>8.85E-05</td>
<td>4.92E-04</td>
<td>2596</td>
<td>9.63E-05</td>
<td>8.36E-06</td>
<td>4.64E-05</td>
<td>2E-05</td>
</tr>
<tr>
<td>sv9-9514</td>
<td>16.40</td>
<td>0.67</td>
<td>0.9</td>
<td>5</td>
<td>4.28E-04</td>
<td>2.38E-03</td>
<td>4000</td>
<td>6.25E-05</td>
<td>2.62E-05</td>
<td>1.46E-04</td>
<td>2E-05</td>
</tr>
<tr>
<td>sv109519</td>
<td>14.28</td>
<td>0.84</td>
<td>0.9</td>
<td>5</td>
<td>3.41E-04</td>
<td>1.90E-03</td>
<td>4500</td>
<td>5.56E-05</td>
<td>1.86E-05</td>
<td>1.03E-04</td>
<td>3E-04</td>
</tr>
<tr>
<td>sv109506</td>
<td>13.4</td>
<td>2.7</td>
<td>0.9</td>
<td>5</td>
<td>1.06E-04</td>
<td>5.90E-04</td>
<td>3000</td>
<td>8.33E-05</td>
<td>8.68E-06</td>
<td>4.82E-05</td>
<td>2E-05</td>
</tr>
<tr>
<td>sv109506</td>
<td>7.425</td>
<td>4.3</td>
<td>0.9</td>
<td>5</td>
<td>6.67E-05</td>
<td>3.70E-04</td>
<td>1700</td>
<td>1.47E-04</td>
<td>9.62E-06</td>
<td>5.34E-05</td>
<td>2E-05</td>
</tr>
<tr>
<td>sv109506</td>
<td>14.38</td>
<td>1.93</td>
<td>0.9</td>
<td>5</td>
<td>1.49E-04</td>
<td>8.25E-04</td>
<td>2400</td>
<td>1.04E-04</td>
<td>1.52E-05</td>
<td>8.43E-05</td>
<td>2E-05</td>
</tr>
<tr>
<td>sv109506</td>
<td>15.38</td>
<td>1.65</td>
<td>0.9</td>
<td>5</td>
<td>1.74E-04</td>
<td>9.66E-04</td>
<td>5300</td>
<td>4.72E-05</td>
<td>8.04E-06</td>
<td>4.47E-05</td>
<td>2E-05</td>
</tr>
<tr>
<td>sv9-9503</td>
<td>18.95</td>
<td>1.83</td>
<td>0.9</td>
<td>5</td>
<td>1.57E-04</td>
<td>8.71E-04</td>
<td>5000</td>
<td>5.00E-05</td>
<td>7.68E-06</td>
<td>4.27E-05</td>
<td>2E-05</td>
</tr>
<tr>
<td>sv9-9503</td>
<td>18.43</td>
<td>1.47</td>
<td>0.9</td>
<td>5</td>
<td>1.95E-04</td>
<td>1.08E-03</td>
<td>8000</td>
<td>3.13E-05</td>
<td>5.98E-06</td>
<td>3.32E-05</td>
<td>2E-05</td>
</tr>
</tbody>
</table>

Notes:
$T$ = Theoretical Time Factor (Torstensson, 1977)
$R$ = radius of cone = 0.01785 m
$c_h$ = Coefficient of consolidation in horizontal direction
$t_{50}$ = time for 50% dissipation of the excess pore pressure
$K_h$ = horizontal hydraulic conductivity
$m_h$ = volumetric compressibility in horizontal direction

305
Calculations similar to Table 10.2 were carried out for all of the sites evaluated as part of the tailings research. However, for the sandier tailings, the dissipation testing approach did not work well at all. Upon investigation, it was shown that there is a limiting value that is approximately between $10^{-4}$ and $10^{-5}$ cm/s. Besides the lack of appropriate theory, the hydraulic conductivity of the filter element of the piezocone is likely a large component in this limiting situation.

10.3.4 Direct Relationships to Mechanical Piezocone Data

One of the most common ways to estimate hydraulic conductivity in geotechnical practice is to simply determine a soil type and then apply a "typical" value from an acknowledged reference. For the most part, such estimates are acceptable for preliminary assessments.

From the predominantly silty and finer tailings at the Sullivan Mine, an attempt was made to investigate such a preliminary estimation technique for mine tailings. The parameter initially chosen to estimate hydraulic conductivity was a simple ratio of normalized cone parameters, $Q/F$. Per the discussions earlier, a pairing of $Q$-$F$ can be related to an $I_c$ value which, as a soil behaviour index, is closely related so soil type. In addition, a strong relationship between $I_c$ and fines content was found to exist.

Figure 10.8 presents the estimated hydraulic conductivities from piezocone dissipation tests with the $Q/F$ ratio. Although there is a very sketchy trend, the relationship is not considered strong enough to be tenable (correlation coefficient, $R^2$, was 0.65).
Figure 10.8  Estimated Dissipation Hydraulic Conductivity versus Q/F - Sullivan Tailings
Another method can be used for estimating the hydraulic conductivity of coarser tailings indirectly from the piezocone. As noted in Section 10.3.3, measurement of pore pressure decay in coarser-grained tailings is more difficult to measure because the excess pore pressures are smaller in magnitude, dissipations occur very rapidly and there is a physical limitation to the piezocone's pore pressure porous element ability to transmit fluid pressure. It is uncertain which theories are valid in this range and the inability to measure these rapid dissipations requires application of other field tests (Section 10.3.6) or empirically based methods. With coarse-grained materials, good empirical correlations exist that relate hydraulic conductivity and relative density ($D_r$) to provide an acceptable estimation technique (e.g. Hunt, 1986). Relative density can be determined directly from the piezocone. A suggested form of the empirical correlations between $K$ and $D_r$ is:

$$\log_{10}(K) = a - b(D_r)$$  \hfill (10.4)

where $a$ and $b$ are empirical constants. For the clean tailings sands in the research database, it was found that an initial estimate of $K$ (cm/sec) can be obtained for $a = 0.5$, $b = 0.05$; $D_r$ is in %. For example, a clean sandy tailings with $D_r = 60\%$, $K$ would be estimated as $3 \times 10^{-3}$ cm/sec. This value is similar to the ~$D_r = 60\%$ clean sand tailings at both Syncrude and the cycloned sand Gibraltar Mine tailings dam. To update values of the constants $a$ and $b$ on a site specific basis, data from laboratory testing or well-tests can be used.
10.3.5 Direct Relationships to Bulk Resistivity Measurements

Another potential method available to predict hydraulic conductivity is to use bulk resistivity values. Research in applied geophysics, particularly related to the petroleum industry, has shown a strong correlation between dielectric soil properties and functional permittivity of rock and soil units (e.g. Knoll et al, 1995).

For the tailings database, the majority of the bulk resistivity data does not allow itself use of relationships such as presented in Knoll et al. (1995) due to the non-standard (e.g. non-freshwater) nature of the pore fluids. For the most part, mine tailings geochemistry tends to invalid the current form of such approaches due to high conductivities in the pore fluids though accommodation for complex pore geochemistry is apparently possible.

10.3.6 KBAT

As noted in Section 4, the KBAT system is a modified form of the BAT hydraulic conductivity device. With the KBAT, 9.53 mm Swagelok valves replace the needle and septum system to allow for much higher rates of flow. This effectively allows measurement of hydraulic conductivities almost two orders of magnitude higher than was previously attainable using the original system.

Initial work with the commercial conductivity BAT and the KBAT utilized vacuum technology. During the test, the progress of pore fluid infiltration to the sampling cylinder could be monitored continuously by calculating the volume of water which has entered the cylinder, based on the pressure recorded by the pressure transducer in the KBAT probe. This calculation was done in
real time by the data acquisition system and is useful, as it allows the operator to determine when the test is complete and the probe can be withdrawn.

The theory used to determine the hydraulic conductivity using the KBAT is based on the work of Hvorslev (1951), that provides the flow equation:

\[ q = F \cdot K \cdot (u_0 - p_t) \]  

(10.5)

where

- \( q \) = fluid flow (m^3/s)
- \( F \) = flow factor (m)
- \( K \) = hydraulic conductivity (m/s)
- \( u_0 \) = static pore pressure at the filter depth (m H_2O)
- \( p_t \) = pressure at any time inside the KBAT cylinder (m H_2O)

Equation 10.5 is a restatement of Darcy’s law, with the ratio of the area to the length of the flow path combined into the flow factor:

\[ F = \frac{A}{L} \]  

(10.6)

The head driving flow into, or out of, the KBAT cylinder is thus the difference between the head outside the drawdown cone surrounding the probe and the pressure inside the cylinder. Since the internal pressure changes with time as water flows into or out of the cylinder, the rate of flow also changes with time.
The use of a constant flow factor is possible since for a given filter geometry this ratio remains constant during the test. For a cylindrical filter with the ratio of length to diameter greater than two, the flow factor can be approximated by the relationship given by Hvorslev (1951), cited in Petsonk (1984):

\[
F = \frac{2 \cdot \pi \cdot L}{\ln \left[ \frac{L}{d} + \sqrt{1 + \left( \frac{L}{d} \right)^2} \right]} \tag{10.7}
\]

where:

- \( L \) = filter length (m)
- \( d \) = filter diameter (m)

In the case of the KBAT filter tip, where the filter length is equal to the diameter, the filter geometry does not satisfy the simplifying assumptions used to derive the above formula. The flow factor was taken from Tavenas et al. (1986). Al-Dhahir and Morgenstern (1969) confirmed the validity of Hvorslev's flow factor with numerical simulations.

The flow factor assumes an isotropic porous medium (\( K_h = K_v \)), which is almost never the case in naturally deposited sediments. However, the derivation of the flow factor assumes that the majority of the flow is horizontal towards the filter, and for mine tailings which almost always would have \( K_h > K_v \), this assumption would certainly be the case. Regardless, any error in the flow factor would have a small impact on computed results, as the flow factor only varies by a factor of about three for practical filter geometries. This is easily within the range of confidence for estimating hydraulic conductivities.
Boyle's ideal gas law is the other fundamental principle used to derive an expression for the hydraulic conductivity based on the KBAT test. This relates the volume of air inside the collection cylinder to the pressure change as measured by the pressure transducer. The resulting formula is given by Torstensson (1984):

\[
K = \frac{p_o \cdot V_o}{F \cdot t} \left[ \frac{1}{u_o \cdot p_o} - \frac{1}{u_t \cdot p_t} + \frac{1}{u^2_o} \cdot \ln \left( \frac{p_t \cdot p_o - u_o}{p_o \cdot p_t - u_o} \right) \right]
\]

(10.8)

where:

- \(p_o\) = pressure at the data point previous to the time of interest (m H2O)
- \(t\) = time increment between two consecutive data points (s)
- \(p_t\) = pressure at the time of interest (m H2O)

This formula does not take into account the pressure head due to the column of water inside the KBAT probe, which changes during the test. During research by the ISTG (e.g., Wilson and Campanella, 1997), however, this effect was found to be negligible for pressure differences between the KBAT cylinder and the surrounding pore water greater than 5 metres of water. Equation 10.8 has been applied by others to the pressure at the beginning of the test and the pressure at any time during the test which has the effect of calculating the secant slope of the pressure-time curve. The method recommended here is to apply the formula between two consecutive data points, which has the effect of calculating the tangent slope.

Companion research during 1995 and 1996 demonstrated that while the KBAT was a vast improvement for obtaining water samples in a high TDS environment when compared to the hypodermic needle commercial BAT system, it was not an effective tool for hydraulic
conductivity measurements in the in-flow (vacuum) configuration. Clogging of the porous filter was common at the high rates of inflow.

Wilson and Campanella (1997), using the same analytical theory noted above, verified that the KBAT would work in sandier materials in an out-flow mode. The work showed that with the 50 mm filter, and the 9.83 mm Swagelok valves, the limiting K value was roughly $1 \times 10^{-2}$ cm/s (medium sand) which is as coarse as most mine tailings. An important finding by the ISTG during the field research was that in-flow K testing in the field often gave incorrect and misleading estimates of hydraulic conductivity which were often more than an order of magnitude too low due to fines migrating and plugging the filter.

The outflow test was evaluated at the Kidd 2-calibration site as described by Wilson and Campanella (1997). It was not applied to any coarse tailings sites. However, it was applied to the Sullivan tailings where, as shown in Table 10.3, there was reasonable well test data available for comparison. From a series of 8 outflow tests, an average estimate of hydraulic conductivity of $5 \times 10^{-5}$ cm/s was found to exist in and around an area where the tested values of hydraulic conductivity were closer to $2 \times 10^{-5}$ cm/s. These results also show that reasonable estimates from the outflow test can likely be found for finer-grained tailings. More research with the KBAT outflow test in a range of tailings materials is required.

10.3.7 Anisotropy

Mine tailings are typically highly layered deposits. This layering results from the grain imbrication and gradation segregation processes which occur as part of the hydraulic deposition
used for placement of most mine tailings deposits. As a result of the layering, which can be on both grain and "lift" thickness scales, tailings deposits are typically highly anisotropic with respect to hydraulic conductivity.

A method is proposed for estimating the ratio of $K_h/K_v$ from using piezocone data. The method makes use of the pore pressure dissipation data and the resulting estimated equilibrium pore pressures along a section of a tailings impoundment. As noted in Section 10.2, this method will not be as successful where very high downward gradients exist as the equilibrium pore pressure values may be roughly equivalent to the elevation head leaving a non-unique solution. The method also assumes a constant tailings material type with depth and a free seepage face (such as the downstream face of a tailings dam). These assumptions are relatively valid for any beached tailings impoundments.

The proposed method assumes that pore pressure dissipation tests are carried out though a section of interest in a tailings impoundment. A minimum of two soundings and preferably at least four, are required to use the method. For such a section in two-dimensions, say the x-z plane, the steady state flow equation is a simplification of Equation 10.1 as follows:

$$\frac{\partial}{\partial x} (\partial h/\partial x) K_x + \frac{\partial}{\partial z}(\partial h/\partial z) K_z = 0 \quad (10.9)$$

The total head (pressure and elevation) at any point on the section is $h(x,z)$ is the solution of Equation 10.9. The piezocone provides the information to describe this solution and back-calculate (estimate) the ratio $K_x/K_z$ ($K_h/K_v$). The proposed method includes defining two angles, A and B; as follows and as shown on Figure 10.9:
1. The dissipated pore pressures with depth for each sounding (either wholly or extrapolated values) provide a slope of total pressure head per unit of depth, \( z \), along the profile of piezocone soundings. The slope of this line for each sounding can be described by \( \tan^{-1}(\partial h/\partial z) = A \) where \( A \) is the angle from the axis of the piezocone sounding (i.e. \( z \)) to the line of equilibrium pore pressures. It is important equal \( x \) and \( z \) scales are used in computing the value of \( A \).

2. A line is created which joins all the points of zero pressure head from the pore pressure dissipation tests on a given section; i.e. the phreatic surface. The slope of this line is \( \partial z/\partial x \) and the numerical slope at any intermediate point (e.g. between two piezocone locations) is \( \tan^{-1}(\partial z/\partial x) = B \).

With the two angles \( A \) and \( B \) defined, Darcy's Law can be used to create an approximate estimate of the \( K_h/K_v \) anisotropy. The line defined by the angle \( B \) is also the top flow line and, along flow lines, Darcian flow velocities \( v_x \) and \( v_z \) are related by:

\[
\tan(B) = v_z/v_x
\]  

(10.10)

The total head is a sum of the pressure head and the elevation head. Along the top flow line, by definition, the pressure head is zero. Consequently, the total head is equal to the elevation head, say \( E \). Therefore, on the top flow line/phreatic surface, the change in head per unit of horizontal distance is equal the tangent of \( B \) as well as to the change in \( E \) per unit of horizontal distance per Equation 10.11:
Figure 10.9  Illustration of Concept of Simplified Piezocone Hydraulic Conductivity Anisotropy Estimation Method
\[
\frac{\partial h}{\partial x} = \frac{dE}{dx} = \frac{dz}{dx} = \tan(B) \tag{10.11}
\]

Darcy's Law is applied in both the x and z directions to yield:

\[
v_z = K_z \cdot \frac{\partial h}{\partial z} \tag{10.12}
\]

\[
v_x = K_x \cdot \frac{\partial h}{\partial x} \tag{10.13}
\]

By combining Equations 10.12 and 10.13 and substituting for A and B, Equation 10.14 results which presents the proposed method:

\[
\frac{K_x}{K_z} = \frac{\tan(A)}{\tan^2(B)} \tag{10.14}
\]

A limiting case of Equation 10.14 would be isotropic conditions; i.e. \( K_x = K_z \). In this case, A and B would both be equal to \( 45^\circ \) which, for A, is reasonable (hydrostatic condition) and, for B, acceptable for the slope of the zero pressure surface as it exits from the tailings dam.

This proposed method was evaluated at both the Sullivan Old Iron Pond and at the Endako Pond No. 1 near the free face of a tailings dam. Table 10.3 presents the results of the evaluation.
Table 10.3  Results of Proposed Method for Estimated K Anisotropy

<table>
<thead>
<tr>
<th>Piezocone Sounding</th>
<th>Dissipation</th>
<th>Phreatic Surface</th>
<th>Estimate of $K_h/K_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Equilibrium Slope, A</td>
<td>Slope, B</td>
<td></td>
</tr>
<tr>
<td>SV95-1</td>
<td>27</td>
<td>3</td>
<td>185</td>
</tr>
<tr>
<td>SV95-2</td>
<td>24</td>
<td>4</td>
<td>91</td>
</tr>
<tr>
<td>SV95-3</td>
<td>29</td>
<td>3</td>
<td>202</td>
</tr>
<tr>
<td>SV95-4</td>
<td>22</td>
<td>3</td>
<td>147</td>
</tr>
<tr>
<td>ENDK96-1</td>
<td>17</td>
<td>18</td>
<td>2.9</td>
</tr>
<tr>
<td>ENDK96-2</td>
<td>19</td>
<td>15</td>
<td>4.8</td>
</tr>
<tr>
<td>ENDK96-3</td>
<td>18</td>
<td>19</td>
<td>2.7</td>
</tr>
</tbody>
</table>

The values for the Sullivan tailings appear reasonable considering the highly oxidized layers from progressively buried beaches, which were exposed and then buried by laminated fine-grained tailings. The predominantly horizontal plane flow to the tailings in the Old Iron Pond area and the very slow infiltration rates experienced at this impoundment are also consistent with the estimated anisotropic ratios of hydraulic conductivity. The Endako values are also reasonable for a more massive and non-oxidized tailings dam that has not seen any formal mechanical compaction. Both values are reasonable when compared to literature values (Vick, 1990).

10.4 Degree of Saturation

In cohesionless soils such as most mine tailings, the bulk resistivity measured is dominated by the pore fluid chemistry. As such, the degree of saturation should follow a fairly basic relationship in these materials with the fully saturated material representing the bulk resistivity measured.

As discussed in Chapter 4, the most common manner to relate bulk and pore fluid resistivities is using the relationship developed by Archie (1942). Archie's equation (1942) is of the form:
Where \( F \) is the formation factor and is the ratio between the bulk and fluid resistivities, \( \rho_b \) and \( \rho_w \) respectively. What controls \( F \) is the porosity, \( n \), in the form:

\[
F \propto n^{-m}.
\]  

Here \( m \) is an empirical constant for a given material called the cementation factor. For most unconsolidated sediments, \( m \) values of 1.3 give good empirical results whereas heavily cemented soils see \( m \) values closer to 2.5 (Archie, 1942).

An alternative form of the formation factor can be given as:

\[
F = a(n)^m(S_r)^s
\]  

where:

- \( a \) = proportionality constant, typically 1.0;
- \( S_r \) = degree of saturation of the porous media; and
- \( s \) = saturation exponent which ranges from 2.0 for water wet conditions (including aqueous phase mixtures) to approximately 10.0 for oil wet conditions (fully non-aqueous phase).

Consequently, using water wet conditions, Figure 10.10 represents the relationship between \( F \) and \( S_r \). A typical tailings porosity of 0.50 was used in developing Figure 10.10. Clearly for use on a specific project, the benchmark value of \( F \) should be established from fully saturated conditions where both \( \rho_b \) and \( \rho_w \) are as representative of the tailings unit of interest as possible.
Once this benchmark is established, significant increases in F can denote areas of unsaturated tailings.

Site specific validation of the proposed relationships in Figure 10.10 was not carried at as part of the research.
Figure 10.10  Estimation of Degree of Saturation from Formation Factor

Saturation Exponent, $s$, = 2.0 (water wet)

Cementation Factor

- $m = 1.3$
- $m = 1.6$
- $m = 1.9$
- $m = 2.3$
11. GEOCHEMICAL NATURE OF PORE FLUIDS

11.1 General

At the outset of the research, a literature review was conducted to examine three basic issues:

- What was the state of piezocone technology with respect to geochemical screening?;
- What was the state of related geophysical fields with respect to geochemical screening with particular emphasis to mine tailings?; and
- What was the state of literature regarding use of piezocone technology for assessing the geochemical nature of mine tailings?

From the review, it was apparent that piezocone, or more accurately the resistivity piezocone and similar devices, were roughly be able to assist in geochemical screening by about the early 1980's. More extensive literature, and some application literature from assorted natural and industrial sites, started to appear in the early 1990's. Campanella and Weemees (1990) describe the UBC efforts in this area and present one of the earliest publications with data from a variety of industries that included natural site materials and their resistivity piezocone characteristics.

The geophysical literature, mainly involving surface electro-magnetics, has sparse references to geochemical screening in the late 1970's through the mid 1980's. By the late 1980's and early 1990's, some of the first literature appeared on using geophysical techniques for delineating, for example, the changes in aquifer characteristics from ARD (e.g. Ebraheem et al. 1990). Recently publications regarding use of geophysical techniques in geochemical screening have become more common.
However, specific references to use of resistivity piezocone technology for mine tailings geochemical work could not be located. At the beginning of this research (1992-93), it was concluded that the efforts of the Author were likely initial efforts and, as such, there would be little technical precedence for comparative purposes. The literature on this subject remains limited and there is substantial need for more research in the area of geochemical screening with piezocone technology to substantiate any of the possible relationships offered later in this Chapter.

11.2 Resistivity as an Indicator of Geochemistry

As described in Chapter 4, the resistivity piezocone provides essentially continuous readings of bulk resistivity of the subsurface during a piezocone sounding. The pore fluid geochemistry, particularly in saturated materials, dominates the resistivity response. More accurately, there is an essentially linear relationship between bulk conductivity (resistivity \( -1 \)) and total dissolved solids within the pore fluids (Keys, 1989).

Pore fluids in natural ground and mine tailings are essentially a solvent. This solvent can have a range of characteristics that will influence its ability to liberate and retain dissolved cations and anions. These characteristics include fluid temperature and pH. The dissolved ions provide the pore fluid with an ability to carry electric charges whereas pure (deionized) water is a perfect insulator. The resistivity piezocone sends an electrical current into the ground. The degree to which the ground (i.e. pore fluid) can carry that current indicates whether the ground is more of an insulator, like pure deionized water, or more of a conductor like, for example, sea water.
As the make-up of pore fluid geochemistry in mine tailings is quite complex, e.g. many different ions can co-exist at any one time in the pore fluid, specific ion detection with electromagnetic methods has not been widely proposed. This research work concurs with this approach although this research found trends for specific ions which may suggest that global relationships for some types of mine tailings may be tentatively present. Far more contributions to the database will be required before any of the relationships provided in Section 11.5 can be used with any confidence without site-specific calibration to laboratory evaluated geochemistry.

11.3 Data Repeatability

One of the key concerns to users of any geophysical characterization technology is the repeatability of the data produced. For this study, excellent repeatability of the standard piezocone channels was evident throughout, as is described in Chapter 4. This repeatability was expected as the trademark of the mechanical channels of piezocone is its repeatability (e.g. Lunne et al., 1997). However, there was some question as to how repeatable the bulk resistivity values would be as this was likely the first application of resistivity piezocone technology to assessment of mine tailings. Figure 11.1 shows a comparison between resistivity measurements from the three electrode spacings (roughly 10, 25 and 75 mm) for one sample resistivity piezocone sounding from the INCO Copper Cliff tailings area; 118-9322 on a bench of the Pistol Dam. Note that even in the upper unsaturated zone of the tailings the agreement is excellent between the channels and, at the most, the difference is a few percentage points in absolute terms. The differences observed are within those expected due to shear-volume changes (dilation effects) during penetration. For this study, such shear-volume coupling effects on resistively
Figure 11.1 Comparison of Bulk Resistivity Measurements for Different Electrode Spacings
results were largely ignored although Section 11.5 following presents an initial proposed
normalization procedure for resistivity piezocone data.

To compare resistivity measurements from different soundings in close proximity in a given area,
several resistivity piezocone soundings were carried out within 10 metres of one another in three
areas at the INCO site. Figure 11.2 shows the results from one such area, Cecchetto Dam, where
some differences above the phreatic surface are evident but well within those expected for
variable saturated conditions. However, below the phreatic surface (depth of approximately 11
metres), the agreement was excellent to the depth that the two soundings were similarly carried
out (17.5 metres).

The good results noted for RES001 were even more impressive with RES002 and RES003. Per
Figure 11.1, several replication trials at the Sullivan tailings area with RES003 showed
repeatability to within a few percent of the actual readings. This level of repeatability is far
greater than that with traditional electrical or electromagnetic tools and is partially due to the very
small volume of material contributing to measurement in comparison with most traditional tools
and the consistent soil disturbance by the resistivity module.
Figure 11.2  Example of Spatial Consistency of Resistivity Piezocone
11.4 Sulphide Mine Tailings

11.4.1 General

Sulphide ore bodies are extremely common. This commonality is particularly true of base metals where anoxic genetic environments are prevalent in the creation of concentrations of economic qualities of many metals. The coal industry is also subject to the presence of sulphide minerals in the sedimentary sequences. With the mining, milling and waste disposal of sulphide ore bodies comes the potential for sulphide mineral oxidation. With mine tailings, and waste rock, this oxidation can lead to significant geochemical alteration to areas adjacent to the tailings.

In the most extreme cases, Acid Rock Drainage (ARD) can develop. ARD refers to the process by which the oxidation of sulphide minerals in the presence of oxygen and water results in the dissolution of formerly bound ionic constituents. The resulting liberated ionic constituents can be ultimately transported to surrounding surficial and groundwater systems, which can have an adverse effect on water quality; particularly when the ions involved are heavy metals and their related complexes. The ARD process and the potential for associated negative environmental impacts at sulphide-based mines have become much better understood over the past 10 to 15 years. Many well-documented cases detailing the effects of ARD exist in the literature (e.g. British Columbia Draft Acid Rock Drainage Technical Guide, 1989).

CANMET (1990) identified the following sources from which ARD generation can occur:

- drainage from underground workings;
- seepage and surface runoff from waste rock piles;
• surface runoff from open pit operations;
• seepage and runoff from tailings impoundments; and
• drainage from ore stockpiles and abandoned heap leach piles.

Mine access roads, railway ballast and other ancillary structures constructed from sulphide rich wastes are also items that have produced ARD and can thus be added to this list. In all of the above cases, the mechanisms involved in generating ARD are essentially the same. These mechanisms involve oxidation of the sulphide minerals present in the overall matrix of the wastes which produces low pH drainage which can mobilize metals and other soluble constituents contained either in the mined materials or in the natural environment. As mine tailings are typically the single largest physical entity to sulphide-ore mining, they often represent the single largest ARD liability. It is generally recognized that once ARD is initiated, costly control and remediation measures are often required in the long-term.

The main concern of ARD is that the resulting metal loading can have negative effects on water quality in receiving environments. In addition, increased sulphate levels, acidity, change in nutrients available and, in some cases, the transport of radionuclides, can also have dramatic impact on aquatic life. As noted in CANMET (1990), in severe cases ARD can and has caused significant loss of fisheries resources due to toxic levels of total dissolved or suspended metal solids.
11.4.2 ARD Processes

This research specifically examined the use of the resistivity piezocone for assessing the effects of sulphide oxidation on mine tailings. As such, to provide completeness, a brief summary of the sulphide oxidation process as related to mine tailings is presented. Far more comprehensive references are suggested for a complete background if the reader is not familiar with the subject.

For acid generation to occur in the context of mine tailings, the conditions that typify the process are:

- the presence of sulphide-based ore leading to sulphide laden tailings materials;
- a source of water or very humid conditions; and
- oxygen.

Complicating the above is that in many instances, bacteria can greatly affect the rate at which the controlling reactions occur. The two main mechanisms of ARD are:

- the flushing of readily soluble contaminants at near neutral conditions; and
- oxidation, acid generation and metal leaching under acidic conditions.

The generation of acidic drainage, its attenuation and its consumption can be quantified by a series of interrelated chemical reactions. For example, at the INCO tailings site, mineralogical analysis indicated that sulphide oxidation of pyrrhotite, pentlandite, chalcopyrite, and pyrite were occurring (Coggans, 1991). At the Sullivan Mine, pyrrhotite and pyrite are the main sulphide
minerals contributing to the oxidation process. The chemical processes for the oxidation of pyrrhotite, chalcopyrite and pyrite respectively can be simplified by the following reactions:

\[ \text{FeS} + 2\text{O}_2 \rightarrow \text{Fe}^{2+} + \text{SO}_4^{2-} \] (11.1)

\[ \text{CuFeS}_2 + \frac{15}{4}\text{O}_2 + \frac{1}{2}\text{H}_2\text{O} \rightarrow \text{Cu}^{2+} + \text{Fe}^{2+} + 2\text{SO}_4^{2-} + \text{H}^+ \] (11.2)

\[ \text{FeS}_2 + \frac{7}{2}\text{O}_2 + \text{H}_2\text{O} \rightarrow \text{Fe}^{2+} + 2\text{SO}_4^{2-} + 2\text{H}^+ \] (11.3)

The ferrous iron (Fe\(^{2+}\)) produced in reactions (11.1) to (11.3) is oxidized by oxygen (O\(_2\)), and subsequently ferric iron (Fe\(^{3+}\)) may react with pyrrhotite as shown in the following chemical reactions.

\[ 8\text{Fe}^{2+} + 2\text{O}_2 + 8\text{H}^+ \rightarrow 8\text{Fe}^{3+} + 4\text{H}_2\text{O} \] (11.4)

\[ \text{FeS} + 8\text{Fe}^{3+} + 4\text{H}_2\text{O} \rightarrow 9\text{Fe}^{2+} + \text{SO}_4^{2-} + 8\text{H}^+ \] (11.5)

\[ \text{FeS} + \text{Fe}^{3+} + \frac{3}{4}\text{O}_2 + \frac{5}{2}\text{H}_2\text{O} \rightarrow 2\text{FeOOH} + \text{S}^0 + 3\text{H}^+ \] (11.6)

The net result of the above reactions is that one mole of H\(^+\) is produced for each mole of pyrrhotite consumed, eventually causing an increase in acidity. Correspondingly, increasing acidity can result in a significantly faster rate of sulphide oxidation.

Bacteria can accelerate the rate at which some of these reactions proceed which in turn increases the rate of acid generation. The bacteria species *Thiobacillus ferrooxidans* can oxidize both reduced sulphur compounds and ferrous iron to ferric iron. Other bacterial species are also
known to catalyze the oxidation of sulphide minerals and these include *Thiobacillus thiooxidans* and *Sulfolobus*. These bacteria derive their energy for existence and cell reproduction from the chemical reaction energy released during the oxidation process.

The overall ARD process is obviously complex and a cursory overview such as given herein is best summarized with Table 11.1. Table 11.1 is a simplified view of the ARD process whereby the process has been split into three separate stages. These stages roughly correspond to pH "plateaus" in the process. These plateaus result from minerals buffering the reaction acidity at various levels of pH throughout the reaction. Figure 11.3 shows a schematic representation of Table 11.1 and shows these plateau phases to the process. Understanding at least the approximate stage of the process is very important in the characterization process.

Another important step in the research was to define what was the nature of real ARD in the mining environment. This step was important so as to understand that whatever response the resistivity module provided, there would be an appreciation of site specific geochemistry. Prior
REATIONS IN STAGE I AND II
FeS₂(s) + 7/2 O₂ + H₂O → Fe²⁺ + 2SO₄²⁻ + 2H⁺
Fe⁺² + 1/4 O₂ + H⁺ → Fe⁺³ + 1/2 H₂O
Fe⁺³ + 3H₂O → Fe(OH)₃ (s) + 3H⁺

pH PLATEAUS RESULTING FROM MINERALS BUFFERING AT VARIOUS pH VALUES

STAGE I

STAGE II

STAGE III

REATIONS IN STAGE III
Fe⁺² + 1/4O₂ + H⁺ → Fe⁺³ + 1/2H₂O
FeS₂(s) + 14 Fe⁺³ + 8H₂O → 15 Fe⁺² + 2SO₄²⁻ + 16H⁺

Figure 11.3  Stages of ARD Development (adapted from Knapp, 1992)
to the field programs, data such as presented in Table 11.2 was gathered so as to prepare the sampling and field monitoring capabilities accordingly; i.e. preparation for low pH and high TDS geochemical environments. These evaluations were carried out at each site using a combination of resistivity piezocone soundings and discrete depth water sampling and subsequent laboratory geochemical analyses.

### Table 11.2 Examples of Acid Rock Drainage Quality (adapted from Fytas et al., 1992)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Seepage from Abandoned Uranium Mine Tailings Pond in Ontario</th>
<th>Waste Rock Dump Seepage from Active Silver Mine in British Columbia</th>
<th>Minewater from U/G Copper Mine in British Columbia</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>2.0</td>
<td>2.8</td>
<td>3.5</td>
</tr>
<tr>
<td>Sulphate</td>
<td>7,440</td>
<td>7,650</td>
<td>1,500</td>
</tr>
<tr>
<td>Acidity</td>
<td>14,600</td>
<td>43,000</td>
<td>--</td>
</tr>
<tr>
<td>Iron</td>
<td>3,200</td>
<td>1,190</td>
<td>10.6</td>
</tr>
<tr>
<td>Manganese</td>
<td>5.6</td>
<td>78.3</td>
<td>6.4</td>
</tr>
<tr>
<td>Copper</td>
<td>3.6</td>
<td>89.8</td>
<td>16.5</td>
</tr>
<tr>
<td>Aluminium</td>
<td>588</td>
<td>359</td>
<td>--</td>
</tr>
<tr>
<td>Lead</td>
<td>0.67</td>
<td>2</td>
<td>0.1</td>
</tr>
<tr>
<td>Cadmium</td>
<td>0.05</td>
<td>0.5</td>
<td>0.143</td>
</tr>
<tr>
<td>Zinc</td>
<td>11.4</td>
<td>53.2</td>
<td>28.5</td>
</tr>
<tr>
<td>Arsenic</td>
<td>0.74</td>
<td>25</td>
<td>0.05</td>
</tr>
<tr>
<td>Nickel</td>
<td>3.2</td>
<td>8.0</td>
<td>0.06</td>
</tr>
</tbody>
</table>

*Note: Other than pH, all other measurements are in mg/litre*

#### 11.4.3 Resistivity Piezocone in Sulphide Tailings Environments

##### 11.4.3.1 General

This portion of the research was focused on evaluating if, and how well the resistivity piezocone could assess geochemical trends in sulphide tailings. To evaluate any trends in the most straightforward manner, reference to Table 11.1 and Figure 11.3 was used to plan a strategy for evaluating performance of the resistivity piezocone during field programs. From Table 11.1, in the most simplistic terms, oxidation of sulphide materials should increase overall ionic loading (e.g. increase pore fluid and hence bulk conductivities above background values), increase
specific metal loading in the pore fluid, increase the sulphate levels in the fluids and show a
decrease in the pH.

The one consistent aspect of the three stages was the presence of the sulphate anion, even at
neutral and above levels of acidity. Consequently, there was immediate attention drawn to need
to evaluate the potential for the bulk resistivity values to assess at least relative values of sulphate
concentration.

Following are very brief summaries of the results of site specific assessments of sulphide tailings
from the research programs. Reference to the summary documents developed by the Author for
the MEND and SCBC projects, UBC-ISTG (1994d) and UBC-ISTG (1997a) respectively, are
recommended for more detailed descriptions of each program. These documents include all
resistivity piezocone soundings and listings of all pore fluid samples and subsequent geochemical
analyses. The quality assurance and control measures for pore fluid handling are also described
in these references. Commercial and the UBC Environmental Engineering laboratories were
used for the ion specific testing. Most of the laboratory testing was completed using Induced
Coupled Plasma (ICP) techniques.

The following summaries are intended to provide an indication of typical results in like tailings
materials as well as show the large range in characteristics for several sulphide mine tailings.
Global (e.g. data from all sites) relationships with specific ions and the resistivity piezocone
response are presented in Section 11.5. To provide some idea of the type of background to such
global relationships, a few sample plots are presented for the INCO tailings.
11.4.3.2 INCO Tailings

For the entire database from the INCO site presented in UBC-ISTG (1994d), the increase in bulk and fluid conductivities are evident in all areas where oxidation effects are likely occurring. Typical characteristics of resistivity piezocone soundings are presented in Appendix I. The actual relationship between the bulk resistivity values and the pore fluid conductivity values (converted to resistivity values) is shown on Figure 11.4. This is some typical data from across all areas of the Central Tailings Area at INCO and includes both tailings and native materials. The clear linear trend demonstrates the confidence with which one can assume that trends in the bulk resistivity are indeed trends in pore fluid conductivity (resistivity). The slight scatter on Figure 11.4 can be largely explained by soil type variation and normalization of the data for soil type can improve the trend even further as described later in this Chapter. It is also important to note that the formation factor (F) values for saturated tailings are quite low when compared to natural soils (typically F=3.5 to 4) indicating the relatively high conductivity of the tailings mineral grains.

Figure 11.5 shows the site wide relationship at INCO for sulphate concentration in the pore fluid versus resistivity piezocone bulk conductivity measurements. Without any attempt to normalize the data, a very good trend is apparent. There appears to be a baseline value in the bulk conductivity that correlates to quite high resistivities for the instrument used (RES001). It is also possible that the values of less than about 200 \( \mu S/cm \) indicate the baseline response of the tailings and native soils in the area.
Figure 11.4  Bulk versus Pore Fluid Resistivity - INCO Tailings
Figure 11.5  Sulphate Concentration versus Bulk Conductivity - INCO Tailings
Figure 11.6  pH versus Bulk Conductivity - INCO Tailings
Figure 11.6 shows the non-normalized trend of resistivity piezocone bulk conductivity with pH for the Central Tailings Area and native soils. A very loose linear trend is evident. The likelihood of a global relationship being present for pH is considered low.

Figures 11.7 and 11.8 inclusive show non-normalized resistivity piezocone conductivity values versus pore fluid concentrations of nickel and iron respectively. The purpose of these comparisons is not to show a specific correlation between specific metal concentrations and bulk resistivity measurements, but rather to discern if there are any site-specific generalized trends. Each of the metals increase in concentration with increasing conductivity.

11.4.3.3 Falconbridge Tailings

From the Falconbridge Fault Lake site, no specific geochemical data was available. However, interesting trends in the measured results were nonetheless evident. Both of the soundings (F01-9333 and F02-9334), as presented in Appendix I, measured zones of extremely low bulk resistivity which are due to the effects of oxidation. The most interesting finding from these soundings was the resistivity measurements in the first five metres of the tailings. Although the tailings in this region are a substantial distance above the prevailing groundwater surface, bulk resistivities below 1 Ω-m were measured, indicating both a very high degree of tension saturation of the tailings and very high ionic concentrations in that pore fluid. For purposes of comparison, the resistivity of drinking water is typically greater than 15 to 20 Ω-m with bulk resistivities, as measured by the resistivity piezocone, being a factor of approximately 2-3 times greater (this is dependent upon the porosity and density of the tailings). It is apparent from the resistivities measured in the upper 5 m that significant sulphate and metal loadings exist, and this is most
Figure 11.8  Iron Concentration versus Bulk Conductivity - INCO Tailings
likely due to the processes of ARD. That the values in the unsaturated zone are higher than the highest saturated values determined at INCO's Central Tailings Area indicates that ionic loading is much higher at the Fault Lake site and more similar to that at the Sullivan Old Iron Pond described later in this Chapter.

There is an additional interesting aspect of the good response in the unsaturated tailings. The practical implication of being able to characterize the resistivity nature of unsaturated media in itself is of considerable merit. Conventional surface and borehole geophysics would most certainly have difficulty due to the problems of maintaining a closed electrical circuit. However the resistivity piezocone appears to be able to avoid this circuit problem with a combination of very small electrode spacings compared to conventional techniques (e.g. mm's versus up to several m's) and the intimate contact that the tool has with the ground due to its method of insertion; no pre-drilled hole is used.

11.4.3.4 Sullivan Tailings

The Sullivan Tailings area and the INCO Tailings area combined provide more than 60% of the overall piezocone data for the research project. In the case of the Sullivan Mine, three separate field programs were completed as noted in Chapter 5.

Following are some brief area summaries describing the use of the resistivity piezocone in the various sulphide-based or influenced tailings at the Sullivan Mine:
Calcine Tailings Area

The Calcine deposits are not actually tailings. Calcine is a product derived from roasting iron pyrrhotite which includes both magnetite Fe₃O₄ and hematite Fe₂O₃; it is clear Calcine deposits are extremely iron rich.

In this area, more than 20 resistivity piezocone soundings were carried out over the three field seasons (1994, 1995 and 1996). The Calcine tailings area was characterized as a very uniform material in the predominantly silt to clayey silt grain size. This area showed a phreatic level decreasing from about 2 metres below the surface to more than 6 metres below the surface from north to south. Both Ic determinations and the pore pressure dissipations corroborated the measured material gradation of clayey to sandy silt.

The lower bulk resistivity values showed a trend of increase from about 1 to 2 Ω-m from north to south. The implication of this trend is a reduction in pore fluid conductivity or, more generally, is typical of a ionic plume with a source at or beyond the northern boundary of the area. This trend is shown in more detail in Chapter 12.

Southwest Limb of Old Iron Tailings Impoundment

This area is characterized at least two distinctly different materials have been placed in this area during the life of the mine. Low resistivity (high conductivity) pore fluid appears to be more prevalent in the upper tailings from beneath the phreatic surface to roughly the transition with the lower material. The values of bulk resistivity to about 0.5 Ω-m are extremely low and indicate
high ionic loading. There is a very pervasive and likely migrating, in downward and southerly
direction plume of ionically laden pore fluid.

There is a downward gradient of between 0.02 and 0.05 in this area with a high level of
anisotropy in the hydraulic conductivity of the lower tailings unit.

**Old Iron Tailings Pond**

More than 10 soundings were carried out in the Old Iron tailings pond. This area provided some
of the most extreme (low) values of bulk resistivity experienced by the UBC-ISTG and/or located
in literature as explained below.

The original resistivity modules developed for this project (RES001 and RES002) were intended
for materials with bulk resistivities from roughly 0.5 ohm-m to more than 1000 ohm-m; the range
of highly ionically contaminated soils to organically contaminated soils. From over 150
soundings in a wide variety of industrial and natural (e.g. salt water) contaminated conditions, no
values lower than 0.5 ohm-m had been experienced.

In the Old Iron Tailings area, values of bulk conductivity lower than 0.05 ohm-m were measured.
Development of RES003 coupled with special calibration and excitation voltage considerations
were required for measurement purposes. The pore fluid in this area is considered as heavily
ionically loaded as experienced by resistivity piezocone experience and is likely close to a
practical upper bound limit for total dissolved solids (at typical groundwater temperatures of 5°
to 8° C). The highest level of TDS measured was nearly 1x10^6 mg/l.
11.4.3.5 Gibraltar Tailings

Quite the opposite of the Sullivan Mine, the pore fluid from the Gibraltar tailings was quite "clean" and the resistivity piezocone reflected this condition. Typical bulk resistivity values were between 10 and 50 Ω-m. The values of all key ions, TDS and the sulphate anion were all essentially the same with copper values of about 0.5 to 1.0 mg/l, TDS around 2700 mg/l and sulphate at approximately 900 mg/l or about one third of the TDS.

Based upon the resistivity piezocone program and tailings pore fluid sampling and testing, the Gibraltar tailings have not undergone any significant sulphide oxidation and the interstitial water is relatively low in total dissolved solids for a base-metal operation.

11.4.3.6 Endako Tailings

Like the Gibraltar tailings, the resistivity character of the molybdenum tailings at the Endako mine were consistent with quite "clean" pore fluid. The results of more than 30 pore fluid samples confirmed this resistivity suggestion. The SO$_4^{2-}$ concentrations were very consistent with a range from 825 to 1625 mg/l. There were several samples with elevated molybdenum and/or cadmium (e.g., ~20 and 10 mg/l, respectively), but overall TDS levels were relatively low (e.g. less than 2000 mg/l). Bulk resistivity values in saturated conditions were typically around 10 to 15 Ω-m.

11.4.3.7 Summary

At each site, irrespective of resistivity level, there was a clear trend of bulk resistivity versus the sulphate loading. This trend was also apparent for other ions. However, the most valuable
aspect of the research may be in the fairly wide variety of character encountered and the reasonable response of the resistivity piezocone technology in each case.

11.5 Global Relationships

All of the resistivity piezocone-pore fluid geochemistry information available from the research tailings database was assembled to see if any global relationships could be tentatively established. As noted earlier, the Author is not a proponent of such relationships until substantially more information is available to validate such relationships.

In each case, dual scale logarithm plots are utilized. There was an effort made to not use such scaling as log-log plots have a tendency to create trends in data where no such trends rightfully exist. However, due to the several orders of magnitude that both ion concentration and bulk resistivity (conductivity) varied across the database, log-log plotting was the only sensible approach. The use of bulk conductivity versus bulk resistivity on many of the plots is purely for ease of comparison to other literature as most of the geochemical information published is in terms of mS/m or µS/m. It is noted that bulk resistivity is a simple reciprocal, with appropriate scaling factors for desired units, to bulk conductivity.

Comments related to drinking water quality are as related to the Guidelines for Canadian Drinking Water Quality.
Total Dissolved Solids

Total Dissolved Solids (TDS) is a measure of all particulate materials that have been solubilized into the pore fluid. Drinking water quality guidelines point to maximum acceptable levels of about 500 mg/l although this is largely an aesthetics criterion.

The TDS for each pore fluid sample was determined in the field with a portable measurement unit prior to placing the sample in a pre-acidified storage container for cold storage (4°C) prior to transport to the testing laboratory. The use of the field metre is not considered as accurate as laboratory determinations. Figure 11.9 shows a definite trend of increasing bulk conductivity with increase in TDS. There is substantial scatter in the data that is considered to be largely due to inaccuracies in the measured TDS values as well as variations in F due to changes in porosity and, hence, amount of pore fluid present in each test case. This latter issue is responsible for some portion of the scatter in each of the plots presented in this Section of the thesis. Section 11.6 addresses the degree to which this scatter can be directly attributable to density (porosity) variations.

Sulphate Concentration

As noted earlier, establishing a relationship with the sulphate anion was considered very important due to its prevalence at all stages of the sulphide oxidation process. High levels of sulphate on their own are not of particular environmental concern although the maximum acceptable values for drinking water are in the range of 500 mg/l with 150 mg/l being the common “taste” threshold. Elevated concentrations of sulphate in drinking water can lead, typically at worst, to gastrointestinal discomfort.
Figure 11.9  Total Dissolved Solids versus Bulk Conductivity - Tailings Database
Figure 11.10 shows the tailings database relationship between bulk resistivity and sulphate concentration. This relationship was one of the better established and the one that the Author considers as close as any to a truly tenable global relationship (Robertson et al., 1997). Included on Figure 11.10 is data from Merkel (1972) from Eastern U.S. coal mining. The ability to be inclusive of the one of the few literature values available lends credibility to the potential global nature of the relationship in Figure 11.10.

An interesting aspect of Figure 11.10 is the bi-linear appearance with a "hinge" at about 1 Ω-m. At this level of bulk resistivity, the pH was typically quite low and this data was solely from areas of present or historic ARD. The most likely explanation for this hinge is the dominance that the $SO_4^{2-}$ anion has as a percentage of TDS at the early stages of the sulphide oxidation process but, as the process proceeds, other ions become increasingly concentrated and the $SO_4^{2-}$ concentration, in relative terms, is somewhat reduced.

**Zinc Concentration**

The maximum 5 mg/l of zinc permitted in approved drinking water is based on aesthetic considerations. Zinc is not considered toxic although, at higher concentrations, it can tend to make water opalescent and create an astringent taste.

As would be expected, the highest zinc concentrations shown on Figure 11.11 are from the Sullivan tailings. There is substantial scatter to this data although there is a trend with higher zinc concentration showing a higher bulk resistivity response. Both the Gibraltar and Sullivan test results showed higher than acceptable zinc from a drinking water perspective.
Copper Concentration

Copper can lead to health concerns at very high levels of ingestion. The acceptable drinking water limit for copper is 1 mg/l. The relationship shown in Figure 11.12 shows a very wide scatter but loose trend of increasing bulk conductivity with copper concentration. The potential for a global relationship is doubtful, as there is a separate clear trend for the Gibraltar data from either the Endako or Sullivan information.

Magnesium Concentration

Magnesium is usually not considered a substance requiring drinking water quality limits. Figure 11.13 shows a reasonable relationship between magnesium and bulk conductivity. The relationship is not overly surprising given the sulphate relationship. Magnesium and sulphate are closely related in the oxidation process and magnesium concentration could also be a potential indicator of potential oxidation production.

Iron Concentration

Figure 11.14 shows the data for iron concentration versus bulk conductivity from the four mine sites where such data was available. A reasonable trend was present over a range of iron concentration from just over 0.1 mg/l to more than 50,000 mg/l.

The drinking water acceptance for iron is 0.3 mg/l which clearly shows that very little of the water sampled for this project would be acceptable on this basis alone. The more objective basis for limitation (mainly on grounds of taste and clothing staining in washing processes) is 0.05 mg/l.
Figure 11.12 Copper Concentration versus Bulk Conductivity - Tailings Database
Figure 11.13  Magnesium Concentration versus Bulk Conductivity - Tailings Database
Figure 11.14 Iron Concentration versus Bulk Conductivity - Tailings Database
Nickel Concentration

Nickel has no specific guidelines as its concentration pertains to acceptable drinking water quality. Figure 11.15 shows the research's limited nickel database. Far from a reasonable trend (except perhaps the Gibraltar data in isolation), there appears to be near invariance between bulk conductivity character and the corresponding level of nickel in the pore fluid. The low concentrations of nickel in comparison to the other ions make it an unlikely candidate to have an impact on the overall bulk conductivity and, as such, the lack of correlation should not be entirely unexpected.

Cadmium Concentration

Cadmium is generally accepted as toxic at high ingestion levels. Maximum acceptable levels in drinking water are 0.005 mg/l with the objective criterion being less than 0.001 mg/l. An inverse relationship between cadmium concentration and bulk conductivity is indicated in Figure 11.16. This heavy metal is more prevalent at high pH and this may explain the trend in Figure 11.16. The levels of cadmium detected in the Endako tailings would be considered to be approaching toxic levels.

pH

Figure 11.17 presents the tailings database for pH, as measured in the field with a portable meter (as per TDS), versus bulk conductivity. As expected, as lower pH conditions lead to more oxidation due to the reducing environment, lower pH levels lead to higher bulk conductivity
Figure 11.16 Cadmium Concentration versus Bulk Conductivity - Tailings Database
Figure 11.17 pH versus Bulk Conductivity - Tailings Database
levels. The trend in the data is quite good for the database save the lowest pH readings. The most acidic of the tailings pore fluids, taken from the Old Iron Tailings area at the Sullivan Mine, was likely beyond the reasonable range for the portable meter as natural occurrences of pH below 1.0 - 2.0 are rare and, as such, the instruments are accordingly calibrated.

11.6 Proposed Normalizing Procedure

Archie's equation is an empirical relationship that has been shown to work extremely well where bulk resistivity is dominated by water chemistry and poorly, if at all, when soil chemistry effects become important. Moreover, if the pore fluid is sufficiently "ion-free" in cohesionless soils, Archie's equation can also fail. Fortunately, most natural waters and essentially all mine tailings pore fluids are sufficiently ionized to avoid this concern in cohesionless (e.g. tailings) soils.

Cohesionless soils are by their nature the largest concern for most pore fluid chemistry problems due to their relatively higher hydraulic conductivity. As such, identifying a meaningful normalization procedure is likely more important for these materials.

As noted earlier in this Chapter, for most unconsolidated sediments, Archie equation m values of 1.3 give good empirical results whereas heavily cemented soils see m values closer to 2.5. If we assume m is 1.3 for most applications, then the ratio of variance in the bulk resistivity to pore fluid resistivity will be $n^{1.3}$. In clean sands, values of porosity, n, of 0.4 are indicative of a medium dense state (which is equivalent to a void ratio, e, of 0.67). Using $m = 1.3$, this soil would have then have a formation factor of 3.29. If we assume that the range in typical sand is to have, say $e_{\text{min}}$ and $e_{\text{max}}$ values of 0.4 and 1.0 respectively, then the corresponding range in $F$ is...
from 5.02 to 2.46. This represents a full range of roughly 2 in F. In other words, with no change in the chemistry of the pore fluid, a difference in measured bulk resistivity of up to a factor of 2 can result simply due to porosity changes (void ratio) that exist within the sand in-situ.

With the piezocone, an indirect measurement of in-situ density is available through the state parameter. Alternatively, the relative density can be determined empirically as a function of cone bearing alone stress level. For the initial effort, the proposed normalization involves a correction for the corresponding normalized cone bearing, Q. The measured bulk resistivity is still the "real" value but the normalization is proposed as a reasonable method with which to compare values in tailings (and other cohesionless soils) so that it is mainly a pore fluid chemistry effect that is able to create significant trends in the data.

The proposed procedure to obtain a value of normalized resistivity, NR, is simply per Equation 11.7 a division the bulk resistivity value by Q.

\[
\text{NR} (\Omega \cdot \text{m}) = \frac{\text{bulk resistivity} (\Omega \cdot \text{m})}{Q} \quad (11.7)
\]

Figure 11.18 shows the effect this normalization has on the scattered data in Figure 11.9, the total dissolved solids versus bulk conductivity. Although there is still significant scatter, the \(R^2\) value for a best-fit log-log linear relationship improved from .61 to .85. Like many of the proposed procedures in this thesis, much more data will need to be evaluated with this normalization suggestion prior to determining whether it provides value to the data evaluation process.
Figure 11.18  Sample Results from Proposed Resistivity Piezocene Normalization Procedure
12. RCPTU IN RELATIONSHIP TO SURFACE ELECTRO-MAGNETICS

12.1 General

Although not presented to any extent in this thesis, the SCBC project included over 30 km of surface electromagnetic surveying. The majority of this data was obtained using the EM-31; a single user tool very common in near surface mineral exploration which is gaining popularity for use in the geoenvironmental field. More details on the use of the surface geophysics during the SCBC project can be found in UBC-ISTG(1997a).

One of the concepts proposed during the research was the potential for a direct analogy between the EM-31 and the resistivity piezocone. Both tools provide a measure of bulk resistivity and it was felt that such an analogy could be found, excellent potential for site characterization with these complementary tools would exist. The EM-31 could be used to locate areas of “interest” prior to planning the sounding locations for the resistivity piezocone (much in the way the state screening method proposed earlier in the thesis can be used to guide optimal soil sampling). In its purest sense, the potential dual approach is a good example of the concept of data worth.

At the Sullivan tailings site between 1994 and 1996, the fieldwork included a comparative area of an EM31 survey and over 30 resistivity piezocone soundings. The EM31 survey and the resistivity piezocone soundings of interest to this particular subject were located at the Calcine tailings area. The surface topography of the site, as with most tailings impoundments, was relatively flat which made for an excellent location to carry out the comparison. There was also a lack of environmental interference to the electromagnetic measurements (e.g. power lines).
It was the objective of this component of the research to compare the two types of data gathered:

1. the resistivity piezocone providing vertical bulk conductivity values with depth;
   and

2. the EM31 survey providing 3 to 6 metre depth near-surface conductivity distribution.

The research initiative involved evaluating whether or not a relationship between the two methods could be established and whether through this relationship a two-step process in potential contaminant location could be designed.

12.2 EM31 Data

The EM31 survey covered nearly 4 km and was carried out as to provide a good coverage of the overall Calcine tailings area. A measurement spacing of 5 m was used. Due to the nature of the test, the EM-31 survey could access locations not trafficable by the ISTG vehicle.

12.3 Creating Conductivity Files from RCPTU Files

To make the best use of the available information and to produce the most accurate results a program was written (Appendix II) which reads raw resistivity piezocone GEODAS data files and reports values at selected depths. The resistivity measurements from the 150 mm electrodes on RES003 were used. The program allows for readings at specified depths to be obtained by averaging the values above and below that depth. Up to 11 points can be averaged (27.5 cm). For this evaluation, composites were obtained at 0.5 m intervals and the point above it and below it were used to obtain an average value. The output from PICK.FOR, files with *.RES extension,
contain the location of the hole, the depth of the reading, the hole identity, and the maximum difference between the resistivity readings averaged. This maximum difference value is a good first indication if the averaging process is a valid one for that zone.

To use the data from PICK.FOR in a plotting package, such as SURFER™, and to create layered maps of the resistivity at different depth intervals a second program was developed, LEVEL.FOR (Appendix II). LEVEL.FOR also converts the resistivity data (ohm-m) into conductivity data (mS/m) to be consistent with the EM-31.

The result from using these two programs is a file that contains a composite file with the conductivity of the porous media at a certain depth at the different sounding locations. These files were processed using SURFER to grid, map and create three-dimensional surfaces of composite bulk conductivity.

12.4 Discussion

The correlation between the EM31 survey surface conductivity values and the conductivity measured using the resistivity piezocone at three sample depths can be seen in comparing Figures 12.1 and 12.2. The good agreement was very encouraging although some agreement was anticipated as both tools are measuring similar phenomena in the same porous media. It appears that the signal recorded with the EM31 is a combination of the different layers found in the porous media. The difference in conductivity scale shows the significance of having intimate ground contact with the resistivity piezocone.
Figure 12.1  Composite Resistivity Piezocone Sounding - Calcine Tailings Area of Sullivan Mine
Figure 12.2  Composite EM-31 Surface from Calcine Tailings Area of Sullivan Mine
The best agreement between the two tools was found to exist in the northern portion of the study area. Highs in conductivity found on the surface corresponded to highs found in the underlying areas. In this area the upper layers of the conductivity measured with the resistivity piezocone do not show great correlation but the lower layers, below 3.5 m or 4.0 m, have high values which could be the cause of the signals recorded by the EM31 survey.

With these encouraging preliminary results, it is suggested that a better approach to geochemical investigations at mine tailings sites can be obtained with a dual tool system. If an EM31 survey is completed prior to the resistivity piezocone/water sample program, the EM31 survey can be used to help optimize resistivity sounding and water sampling to areas where the most valuable information could be obtained.

As an aside to the EM-resistivity piezocone conclusions, Figure 12.2 provides an interesting and meaningful contribution to the points made on data worth at the outset of the thesis. Figure 12.2 shows the location of four observation wells (the four-corner approach) placed to determine if there was any potential groundwater contamination concern in the area. From the wells, no concern was ever indicated and a plume was allowed to develop over more than 30 years. Although the original intent of the observation wells is not questioned, the highest data worth for an observation well was clearly not realized by any of the four wells.
13. SUMMARY AND RECOMMENDED FURTHER STUDY

13.1 Summary

13.1.1 General

The stated objective of the thesis research involved demonstrating the degree to which piezocone technology can be utilized in the geoenvironmental characterization of mine tailings. Based upon a fairly substantial database of piezocone, bulk resistivity, seismic wave velocities and pore fluid geochemistry measurements developed from five years of field studies at eight separate sites, several of the main sub-topics within the broad scope of geoenvironmental site characterization were investigated. The database is possibly the largest in existence for piezocone technology related specifically to mine tailings. For each sub-topic evaluated, existing methods of analyses were examined for their applicability to the research database. In many cases, there was either no existing method or little applicability from methods that were created for natural soil deposits. Consequently, several revised or wholly new procedures have been postulated and tested against the database developed. The combination of the new testing tools, the database itself which includes initial efforts such as the application of resistivity piezocone to sulphide mine tailings, and the revised/new methods of data analysis for a wide variety of geoenvironmental issues, represents the original contributions of the research. The linking of the geoenvironmental disciplines of geotechnics, hydrogeology and geochemistry in a single piezocone research program is also unique.

13.1.2 Piezocone Technology

The research included the creation of a “new generation” resistivity module for piezocone sounding that involves isolated circuitry (e.g. stable linear calibration even at very low levels of
bulk resistivity) and the ability to measure bulk resistivity over a range from less than 0.08 Ω·m to more than 1000 Ω·m. The new resistivity module, and supporting data acquisition, also allows induced polarization measurement.

An in-situ water sampling technology was fashioned that built upon a commercially available system with the main improvement being an ability to sample in the high suspended and dissolved solids environment present in the pore fluids of most mine tailings.

The concept of a cyclic piezocone has also been introduced.

### 13.1.3 Tailings Database

An extensive body of in-situ test data has been compiled on a variety of mine tailings. The research included over 200-resistivity piezocone soundings, many with seismic wave data, and more than 100 pore water samples. Due in part to complimentary research programs, e.g. the CANLEX project, this in-situ database is also supported by quality laboratory data.

The database includes, to the Author’s knowledge, the first use of resistivity piezocone technology to evaluate sulphide mine tailings. The resistivity piezocone data was obtained in combination with discrete-depth pore fluid samples that were geochemically analyzed by commercial laboratories.

The overall piezocone, resistivity piezocone and geochemical database was augmented with surface electromagnetic geophysical information at most of the research sites.
The database also provides a contribution to the relatively sparse geotechnical data on non-plastic silts. This contribution includes the introduction of a well-characterized static-liquefaction case history; one of the few such case histories available.

13.1.4 Soil Behaviour Type

Stress normalization is extremely important for mine tailings behaviour classification. The range of stress levels in most tailings storage impoundments is substantial and traditional non-normalized classification procedures tend to show material type changes with depth where there are no such changes in reality.

The proposed method for assessing tailings soil behaviour type includes a single relationship with normalized factors of piezocone tip \( (Q) \), sleeve \( (F) \) and pore pressure \( (B_q) \) folded together. The method represents the first time all of the three independent piezocone parameters have been integrated on a single interpretation chart. Introduction of the concept of a material index, \( I_c \), which is based on \( Q \), \( F \) and \( B_q \), proved to be applicable for both soil behaviour type classification of mine tailings and in relating piezocone data to many geomechanical characteristics of these tailings. Included in the original contributions of the research is a manner of using \( I_c \) to estimate the fines content of mine tailings.

Soil behaviour type is not necessarily equivalent to the soil type from traditional soil classification systems. It is argued that for all practical purposes of dealing with mine tailings, understanding how the tailings will behave is of far greater importance than what the tailings are named.
13.1.5 Geotechnical Parameters

The in-situ state of a unit of mine tailings is the controlling variable to its resulting geomechanical behaviour upon shear loading; static or transient. A modification to an approach originally developed by the Author and his colleagues for natural deposits (Plewes et al., 1992) was shown to provide a better prediction of state than any available methods when compared to reference frozen sample and laboratory measurements of the CANLEX database and research. Moreover, the proposed method helps to explain earlier controversy related to the assessment of the in-situ density of the Beaufort Sea hydraulic fills. The method also effectively evaluates tailings labeled as problem soils by other researchers (e.g. the compressible Alaskan tailings).

The key original contribution of the proposed approach over previous state parameter methods is the inclusion of tailings compressibility. The assessment of compressibility from piezocone data alone is shown to provide good estimates when compared to reference measurements from laboratory tests.

Both static and transient load geotechnical parameters were evaluated and shown to agree with documented reference data. The former, however, were only briefly examined although a workable procedure using \( I_c \) to predict SPT \( N_{60} \) values in tailings from piezocone data without need for obtaining physical samples is introduced.

Some typical values for static geotechnical design parameters are provided from the range of tailings in the research database to act as an initial step where site specific data may be lacking.
Transient loading issues included examining liquefaction susceptibility, typical moduli and damping values and estimating both constrained and unconstrained liquefied strengths of mine tailings. The separation of the undrained liquefied strength into constrained and unconstrained classes may be of assistance in removing some of the confusion in the literature with respect to choosing appropriate design values.

The proposed piezocone method for screening tailing for their liquefaction susceptibility, an indirect method based upon a compressibility compensated state parameter, is shown to provide good results in terms of defining contractant versus dilatant tailings. The results presented to assess this liquefaction screening procedure include a number of well-documented case histories. For example, the proposed liquefaction susceptibility screening method correctly predicted the non-brittle response of the CANLEX J-Pit trial and correctly identified the approximate zone of contractant tailings involved in the 1991 Sullivan Mine static liquefaction dyke failure.

A method for estimating the post-liquefaction strength of constrained tailings (constrained referring to geometric confinement precluding flowslide development) is introduced. The method is based upon a stress normalization approach that is shown to have both a case history and a soil mechanics basis. An original screening level relationship is proposed for the prediction of a constrained \( Su / \sigma'_w \) from piezocone data. This relationship was shown to provide a good estimate of the back-analyzed strength of the 1991 Sullivan Mine failure but under-predicted the likely post-liquefaction strength for the 1978 Moshikoshi tailings dam failure in Japan.
Use of a cyclic piezocone to estimate the undrained strength of tailings was shown to have some potential but was not sufficiently developed by this research.

Concerns were presented regarding the use of shear wave velocity methods for assessing the liquefaction susceptibility of mine tailings. It is suggested that many mine tailings have grain to grain “bonds” which make prediction of large-strain geotechnical behaviour difficult from small-strain seismic wave data.

13.1.6 Hydrogeological Parameters

The physical hydrogeologic system within a mine tailings impoundment is typically quite complex. Proposed methods to estimate in-situ gradients and hydraulic conductivity of tailings using the resistivity piezocone have been presented. The value of the resistivity module in assessing the location of the water table and, consequently, allowing accurate flow modeling is demonstrated. It is suggested that accurate gradient and effective stress calculations in mine tailings cannot be made without this knowledge of the water table location available with the resistivity piezocone. In addition, an original concept of estimating the hydraulic conductivity anisotropy inherent in mine tailings embankments is introduced.

13.1.7 Pore Fluid Geochemistry

This research presents an initial and original effort in assessing the use of resistivity piezocone technology to assess the geochemical character of mine tailings. The application of surface geophysical techniques to augment the resistivity piezocone is shown to have merit and the two tests can be linked by simple analogies to optimize site characterization programs where both
technologies are applied. The geochemical database established allowed several specific ions to be assessed for the ability (at least from the database developed) of the bulk resistivity/conductivity from the piezocone to estimate the pore fluid concentrations of these ions.

The research program included sites heavily affected by sulphide oxidation. These sites can be characterized by very high levels of total dissolved solids in the pore fluids of the tailings. Both pore fluid sampling and resistivity piezocone technologies were originally developed and enhanced during the tenure of the research and were shown to work well for characterizing in-situ conditions in these challenging environments. There appears to be good promise for the resistivity piezocone to estimate sulphate concentration that may eventually lead to an ability to rapidly evaluate mine tailings in-situ for the relative stage of sulphide oxidation processes.

13.2 Recommendations for Further Study

A wide variety of geoenvironmental characterization topics were addressed in this thesis. All of the topics can benefit from additional research. There are proposed concepts that require substantial review by adding to the available database and checking the proposed relationships against an enlarged database. The database assembled, as extensive as it is, involves data from fewer than ten tailings sites. For perspective, there are over 50 active and inactive tailings impoundments of consequence in British Columbia alone.

There are a number of specific recommendations for further research which include:

Tool Development

- further extension of the measurement range of the resistivity piezocone to include an accurate lower bound to at least 0.01 Ω-m;
• further research with the resistivity technology to evaluate the potential for induced polarization, focused resistivity and dielectric constant measurements;

• continued development and assessment of a cyclic piezocone;

• evaluation of additional piezocone sensors for use in the mine tailings environment;

• carry out numerical modeling on resistivity piezocone insertion and develop stress and electromagnetic field analogs consistent with measured results; and

• develop initial ASTM/International standard for resistivity modules.

Tailings Database

• add more variety of tailings, e.g. gradation, plasticity and compressibility, to the piezocone and resistivity piezocone database; and

• augment the in-situ data with geotechnical laboratory testing to expand the proposed in-situ state assessment procedures.

Geotechnical Parameters

• continue to add case histories of statistically or dynamically induced liquefied tailings masses which are evaluated with the piezocone;

• expand and refine the correlation between material index and fines content; and

• develop relationships for key geotechnical parameters in terms of either $\Psi$ or $I_c$ which are derived solely from piezocone data.

Hydrogeological Parameters

• continued assessment of the potential for rapid dissipation technology (e.g. acquisition time-step $\Delta t = 0.001$ sec) for estimating the hydraulic conductivity of tailings;

• continued evaluation of the KBAT in a variety mine tailings; and

• enhance the theory and establish data requirements for estimating the degree of saturation from bulk-pore fluid resistivity measurements.

Geochemical Parameters

• greatly expand this initial work and continue to assess the potential for global correlations between specific ions and bulk resistivity and/or similar geophysical piezocone technology; and
• enhance the proposed normalization procedure for bulk resistivity measurements and eventually develop an acceptable standard for all resistivity piezocone programs.

The mining industry will continue to evolve ways with which to effectively store mined tailings so as to limit the potential for structural or geochemical failures. Regardless of the evolution that may occur, the industry will continue to require procedures to both assess design assumptions and to allow optimization of facilities that are typically developed over many years. Piezocone technology appears well-suited to assist in these assessments of mine tailings facilities.
REFERENCES


ICOLD (1994). Tailings Dam Incidents, a report prepared by the USICOLD committee on Tailings Dams, November.


UBC-ISTG (1997b). CPTINT 5.0 Piezocone Interpretation Software for the IBM-PC, Department of Civil Engineering, University of British Columbia, Vancouver, B.C., Canada, V6T 1Z4


APPENDIX I

Summary Site Descriptions and Sample Data
APPENDIX I

SUMMARY SITE DESCRIPTIONS AND SAMPLE DATA

I-1. FIELDWORK

I-1.1 General

As summarized in Chapter 5, the overall research project included more than 200 piezocone soundings, 100 pore fluid samples and 30 km of surface electromagnetic geophysical surveying. This Appendix presents a summary of the details of each site and some sample data from two of the sites used to establish the tailings research database. Figure 5.1 in the main text presents the location of these sites.

There are more details provided in this Appendix on the MENDO-INCO project than for the other sites in the research database. It was reasoned that to demonstrate the degree of background information and scope of the database developed for each site, such detail was required. At the same time, as this information does not aid in the use of the thesis, the remainder of the sites are only briefly described. More detailed information, including all piezocone soundings, laboratory geochemical reports and site specific conclusions, can be found in a series of UBC-ISTG documents developed by the Author during the research tenure. These documents are listed in the reference list to the main body of the thesis.

I-1.2 MENDO-INCO Project

I-1.2.1 General

In 1993, the initial project in support of the thesis was established with joint funding by MEND Ontario and INCO, Sudbury. The UBC research vehicle was rail transported to Sudbury where the project outlined in Chapter 5 was completed at both the INCO Copper Cliff Tailings Area and Falconbridge’s Fault Lake Tailings Area. These two tailings areas, and typical piezocone sounding data for each, are presented in the below.

I-1.2.2 Inco Copper Cliff Mine, Ontario

I-1.2.2.1 Site Description

General

INCO's Central Tailings Area is located near the town of Copper Cliff, Ontario, just west of the city of Sudbury. The Central Tailings Area consists of six inactive impoundments (designated as A, CD, M-M1, P, and Q) where tailings deposition occurred from 1937 to 1988, and the adjacent R area (comprised of R1, R2, R3, and R4) where tailings are currently being, and will be in the future deposited. In addition, a smaller inactive tailings impoundment exists in the Upper Pond area to the north of the INCO smelter complex. The tailings occupy an area in excess of 20 km², and the depth of the tailings range from less than 1 m, where they lie directly on the bedrock ridge, to greater than 45 m overlying the Pleistocene sediment cover in the centre of the main tailings area (Coggans, 1991).
INCO's Copper Cliff operation processes ores extracted from the Sudbury sublayer. The ore mineralogy of the sub layer is generally well understood, but many minor phases are present (Hawley, 1962). The majority of the Sudbury ore consists of varying proportions of pyrrhotite, pentlandite, and chalcopyrite. Other minor phases of local importance include pyrite, cubanite, and millerite (Pattison, 1979).

**Mineralogy of the Tailings**

It is difficult to give absolute values for the mineralogical constituents of the Copper Cliff tailings due to variations in the host rocks and ore grades processed over time, and changes in milling practices. However, an approximate estimate of tailings mineralogy is as follows (Peters, 1984):

<table>
<thead>
<tr>
<th>Mineral</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Feldspar</td>
<td>50%</td>
</tr>
<tr>
<td>Magnetite</td>
<td>0.6%</td>
</tr>
<tr>
<td>Amphiboles (chlorite)</td>
<td>20%</td>
</tr>
<tr>
<td>Pentlandite</td>
<td>0.5%</td>
</tr>
<tr>
<td>Quartz</td>
<td>10%</td>
</tr>
<tr>
<td>Chalcopyrite</td>
<td>0.3%</td>
</tr>
<tr>
<td>Pyroxenes</td>
<td>7%</td>
</tr>
<tr>
<td>Biotite</td>
<td>7%</td>
</tr>
<tr>
<td>Pyrrhotite</td>
<td>5.6%</td>
</tr>
</tbody>
</table>

From the above list it is apparent that pyrrhotite is the principle contributor to the overall sulphide content of the tailings. The range of sulphide content, by weight, for the majority of the tailings is between 0.5% and 5%.

**Testing Program**

The UBC-ISTG carried out a geoenvironmental field investigation program at INCO's Copper Cliff tailings site in the autumn of 1993. Preparatory site work was initiated on September 29, 1993 and subsequent field-testing followed during the inclusive period October 1 - 14, 1993. The location of all holes were designated by INCO personnel, with the exception of the holes at the base of Pistol Dam which were selected by the UBC-ISTG to optimize geochemical data available from well installations placed and maintained by the University of Waterloo.

A summary of the testing program carried out is given in Table I.1. The numbering system used to designate each piezocone sounding was specific to this research program, and was partially based on the numbering scheme used by INCO for testhole location description. The first three characters in the hole designation give information on field location and the remaining four characters describe the order in which testing was done. For example, IO1-9329 was designated as Hole #1 according to the INCO system and was the 29th piezocone sounding in the overall piezocone program.

The approximate location of each field test with reference to the mine grid coordinates of the Central Tailings Area is given in Figure I.1.
<table>
<thead>
<tr>
<th>Location of Sounding</th>
<th>File Name</th>
<th>Test Date (month/day)</th>
<th>Equipment Used</th>
<th>Sounding Depth (m)</th>
<th>Specific Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>M-M1 Area</td>
<td>IO3-9301</td>
<td>10-1</td>
<td>UBC9/RES</td>
<td>9.625</td>
<td>BAT water samples were recovered at 5.60 and 6.00 m.</td>
</tr>
<tr>
<td></td>
<td>IO2-9304</td>
<td>10-3</td>
<td>HO36/RES</td>
<td>42.300</td>
<td>PAT water samples were recovered at 4.0, 5.0, 6.0, 7.0, and 8.0 m.</td>
</tr>
<tr>
<td>A-Area</td>
<td>IO6-9302</td>
<td>9-2</td>
<td>UBC9/RES</td>
<td>18.200</td>
<td></td>
</tr>
<tr>
<td></td>
<td>IO6-9303</td>
<td>9-2</td>
<td>HO36/RES</td>
<td>18.200</td>
<td></td>
</tr>
<tr>
<td></td>
<td>IO6-9321</td>
<td>9-10</td>
<td>HO36/RES</td>
<td>19.125</td>
<td></td>
</tr>
<tr>
<td>C-D Area</td>
<td>IO4-9305</td>
<td>10-3</td>
<td>UBC9/U2@U3</td>
<td>7.725</td>
<td></td>
</tr>
<tr>
<td>Pistol Area -</td>
<td>II1-9308</td>
<td>10-4</td>
<td>HO36/U2</td>
<td>13.775</td>
<td>Sounding was at crest of dam.</td>
</tr>
<tr>
<td>Pistol Dam</td>
<td>II1-9309</td>
<td>10-5</td>
<td>HO36/RES</td>
<td>40.200</td>
<td>Sounding was at crest of dam.</td>
</tr>
<tr>
<td></td>
<td>II8-9322</td>
<td>10-11</td>
<td>HO36/RES</td>
<td>36.400</td>
<td>Sounding was at first bench of dam (Sta. 386 on existing I.P. line)</td>
</tr>
<tr>
<td></td>
<td>II9-9323</td>
<td>10-11</td>
<td>HO36/RES</td>
<td>6.525</td>
<td>Sounding was at base of dam (4m equidistant from Waterloo piezometer nests IN22, IN21, and IN23)</td>
</tr>
<tr>
<td></td>
<td>II20-9324</td>
<td>10-11</td>
<td>HO36/RES</td>
<td>5.200</td>
<td>Sounding was at base of dam (10m North of Waterloo piezometer nest IN23)</td>
</tr>
<tr>
<td></td>
<td>II21-9325</td>
<td>10-11</td>
<td>HO36/RES</td>
<td>4.950</td>
<td>Sounding was at base of dam (adjacent to Waterloo piezometer nest IN23)</td>
</tr>
<tr>
<td></td>
<td>II22-9326</td>
<td>10-12</td>
<td>HO36/RES</td>
<td>3.875</td>
<td>Sounding was on service road at the base of dam (adjacent to Waterloo piezometer nest IN32)</td>
</tr>
<tr>
<td></td>
<td>II23-9327</td>
<td>10-12</td>
<td>HO36/RES</td>
<td>14.475</td>
<td>Sounding was on service road at base of dam (adjacent to Waterloo piezometer nest IN22)</td>
</tr>
<tr>
<td></td>
<td>II24-9328</td>
<td>10-12</td>
<td>HO36/RES</td>
<td>3.425</td>
<td>Sounding was on service road at base of dam (adjacent to Waterloo piezometer nest IN27)</td>
</tr>
<tr>
<td></td>
<td>II1-9330</td>
<td>10-13</td>
<td>HO36/RES</td>
<td>43.275</td>
<td>Sounding was on service road at base of dam (adjacent to Waterloo piezometer nest IN33)</td>
</tr>
<tr>
<td></td>
<td>II25-9331</td>
<td>10-14</td>
<td>HO36/RES</td>
<td>3.850</td>
<td>Sounding was on service road at base of dam (adjacent to Waterloo piezometer nest IN33)</td>
</tr>
<tr>
<td></td>
<td>II26-9332</td>
<td>10-14</td>
<td>HO36/RES</td>
<td>10.075</td>
<td>Sounding was on service road at base of dam (adjacent to Waterloo piezometer nest IN34)</td>
</tr>
<tr>
<td>Whissel Dam</td>
<td>II2-9306</td>
<td>10-4</td>
<td>HO36/RES</td>
<td>15.050</td>
<td>Sounding was stopped due to excessive inclination</td>
</tr>
<tr>
<td></td>
<td>II2-9307</td>
<td>10-4</td>
<td>UBC9/U2@U3</td>
<td>28.425</td>
<td></td>
</tr>
<tr>
<td>Cecchetto Dam</td>
<td>II0-9313</td>
<td>10-6</td>
<td>HO36/RES</td>
<td>18.375</td>
<td></td>
</tr>
<tr>
<td></td>
<td>II0-9314</td>
<td>10-6</td>
<td>HO36/RES</td>
<td>22.500</td>
<td></td>
</tr>
<tr>
<td>Q-Area</td>
<td>II13-9310</td>
<td>10-5</td>
<td>HO36/RES</td>
<td>7.75</td>
<td></td>
</tr>
<tr>
<td></td>
<td>II17-9311</td>
<td>10-6</td>
<td>HO36/RES</td>
<td>13.700</td>
<td></td>
</tr>
<tr>
<td></td>
<td>II14-9312</td>
<td>10-6</td>
<td>HO36/RES</td>
<td>8.475</td>
<td></td>
</tr>
<tr>
<td>R3 - Dam 15</td>
<td>II09-9315</td>
<td>10-7</td>
<td>HO36/U2</td>
<td>10.575</td>
<td></td>
</tr>
<tr>
<td></td>
<td>II09-9316</td>
<td>10-7</td>
<td>HO36/U2</td>
<td>10.400</td>
<td></td>
</tr>
<tr>
<td>R2 - Dam 14</td>
<td>II08-9317</td>
<td>10-7</td>
<td>HO36/RES</td>
<td>2.075</td>
<td>Sounding was stopped due to extremely dense material</td>
</tr>
<tr>
<td></td>
<td>II08-9318</td>
<td>10-7</td>
<td>HO36/U2</td>
<td>8.450</td>
<td></td>
</tr>
<tr>
<td>Upper Pond</td>
<td>II15-9319</td>
<td>10-8</td>
<td>HO36/RES</td>
<td>19.675</td>
<td></td>
</tr>
<tr>
<td></td>
<td>II16-9320</td>
<td>10-9</td>
<td>HO36/RES</td>
<td>19.250</td>
<td></td>
</tr>
<tr>
<td>Pyrrhotite Dam</td>
<td>II01-9329</td>
<td>10-13</td>
<td>HO36/RES</td>
<td>41.625</td>
<td></td>
</tr>
</tbody>
</table>

Note: The symbols used to designate the equipment used for each sounding are interpreted as follows:
- HOS3 - Hogentogler Co. piezocone
- UBC9 - UBC piezocone
- U2@U3 - Position of pore pressure elements for sounding
- RES - Resistivity module included in piezocone sounding
At the Central Tailings Area a total of 32 soundings were carried out. 10 BAT sample depths at three sounding locations augmented these soundings. In addition, several of the sounding locations at the toe of the Pistol Dam were directly adjacent to University of Waterloo sampling piezometer nests. As a consequence of the University of Waterloo installations, independent water chemistry data was available in this area of the tailings complex.

I-1.2.2.2 Research Program

The main investigation site for the study was INCO's Central Tailings Area. A total of 13 days were spent collecting data in a mainly demonstrative and research capacity (i.e. non-production mode). Notwithstanding the research data acquisition capacity, a total of nearly 550 metres (about 1800 feet) of RCPTU and CPTU soundings and 10 BAT water samples were achieved during the Central Tailings Area program.

I-1.2.2.3 Typical Data

Sounding 106-9321 (A - Area)

This sounding was carried out using a resistivity piezocone. A summary of sounding 106-9321 is shown on Figure 1.2. The upper soil stratum to a depth of approximately 2.6-m consists of dense sand, as indicated by the cone bearing and pore pressure data. This dense material is underlain by a thick layer of sand in a loose to medium dense state. However, a silty-sand lens at a depth of approximately 6.4-m partitions this stratum. The increase in friction ratio, decrease in cone bearing, and the slight generation of excess pore pressure is indicative of a contractive silty-sand material. The native material, compact sand to sandy and clayey till, exists below a depth of approximately 17.7-m. The RCPTU met refusal at a depth of 19.10 m on a very large boulder or bedrock.

The bulk resistivity profile from the outer electrode spacing is also shown on Figure 1.2. The resistivity measurements clearly pick up the water table at a depth of approximately 1.5-m. The resistivities are lowest and remain relatively constant from 3 to 8 m, fluctuating near 10 Ω-m. The BAT system was also used at this location, and a total of five groundwater samples were recovered from 4 - 8 m. The resistivities of the water samples ranged from 1.6 - 2.6 Ω-m.

As this was among the first resistivity piezocone soundings carried out for the research program, there was uncertainty to what kind of results would be obtained. A comparison of bulk conductivity (1/resistivity) measurements to the conductivity of the water samples showed the conductivity of the saturated tailings to be much lower than that of the pore fluid. It was therefore expected that bulk values of conductivity would be significantly lower than the values for the water samples; a finding that was later accepted as the rule with resistivity piezocone testing in sulphide mine tailings. The data, however, show the same trends and the trough in the bulk conductivity at 7 m is well matched by a corresponding decrease in pore fluid conductivity. The agreement between the data in this initial effort in the overall thesis program was encouraging as it was the first to demonstrate the potential effectiveness of the resistivity piezocone in identifying zones of higher conductivity and increased ionic loading in subsurface pore waters of sulphide mine tailings.
Sounding I11-9330 (Crest of Pistol Dam)
This sounding is shown on Figure I.3. There was a very dense oxidized layer evident from the surface to a depth of approximately 3-m. Below this depth, fine sandy tailings were present that showed a slight fining downward trend. Based upon the pore pressure and bulk resistivity response, the phreatic surface was determined to be located at 15 m. However, it is interesting to note that some lower resistivity values (higher conductivity) do exist above the phreatic surface. The moisture retention capabilities of tailings is well-documented and it is clear that the unsaturated tailings in this area still have water contents that are quite close to 100% and that the pore fluid present has elevated level of ionic constituents. Below the phreatic surface, the bulk resistivity values were quite constant at approximately 10 \( \Omega \cdot m \), which was similar to the saturated values found in the lower materials in the A-area. A value of 10 \( \Omega \cdot m \) indicates that some, although not high, level of ionic loading is present in the pore fluid.

The pore pressure profile from 15 m to 37 m in the tailings shows inter-layering of likely slightly contractile and dilatant materials. This signature is consistent with beached tailings that periodically are allowed to establish a desiccated and oxidized layer during the period following deposition and prior to further dam construction.

The sounding exited from the tailings at a depth of about 37 metres into a very stiff silty clay deposit. In this fine-grained native material, excess pore pressures of over 100 m of water pressure were measured during penetration attesting to the very stiff nature, low hydraulic conductivity and high stress levels present in this material. The bulk resistivity values in the native materials rose sharply to values of about 40 \( \Omega \cdot m \) which would indicate that little, if any, ionic loading above expected background values exist in these tailings.

Sounding I19-9323 (Base of Pistol Dam)
This sounding was one of eight RCPTU soundings carried out at the base of Pistol Dam and is shown in Figure I.4. The upper 0.8-m is a relatively firm clay material, and this is well demonstrated by the peak in friction ratio values above 3% to 5% and the response of the pore pressure measurement. This upper layer is underlain by a fine silty-sand to a depth of approximately 1.9-m in thickness. Beneath these strata is dense sand to a depth of 6.5 m where the cone encountered a thin till mantle over a large boulder or bedrock. The negative pore pressures produced during penetration through this layer are due to the dilatant behaviour of the sand, indicating a relatively high in-situ density. The loose sand lens at a depth of 5.6 m is shown by the decrease in cone bearing and the equilibrium pore pressure during penetration. The relative density in this thin zone is approximately 25%. This relatively thin zone is an example of a layer that may have significant geotechnical and hydrogeological implications yet would be missed by coarse interval sampling techniques.

The resistivity profile from the outer electrode spacing is also shown on Figure I.4. The resistivities remain relatively constant throughout the depth, ranging from approximately 7 - 10 \( \Omega \cdot m \). The location of this test was selected such that the hole was 4 m equidistant from three existing University of Waterloo piezometer nests (IN82, IN21, and IN23). Resistivity measurements were made on groundwater samples from IN21 and IN23 during November, 1990 and resistivity measurements on groundwater samples from IN82 were made during July, 1991 (De Vos, 1992). The resistivities of the water samples ranged from 2.4 - 2.9 \( \Omega \cdot m \), and these low
values are due to increased dissolved solids loading, likely from oxidation processes in the tailings.

I-1.2.3 Falconbridge Mine, Ontario

I-1.2.3.1 Site Description

The Fault Lake tailings site is located near the town of Falconbridge, Ontario, and approximately 0.5 km east of the Sudbury Airport. Tailings were deposited at the Fault Lake site from 1965-1978, which encompasses an area of approximately 22 hectares with the tailings being to a maximum depth of approximately 30-m. The creation of the Fault Lake tailings area consisted of filling in several glacial kettle lakes whose remnant deposits form the basal layers for the tailings area.

Mineralogy of Tailings

It was reported that the tailings contain as much as 50% pyrrhotite. The tailings are all products of sulphide ore recovery.

Testing Program

Two RCPTU soundings were conducted at Falconbridge's Fault Lake tailings impoundment on the afternoon of October, 14, 1993. The approximate locations of the two soundings are shown on Figure 1.6.

I-1.2.3.2 Research Program

Two RCPTU soundings were carried out at the Fault Lake Tailings site. This site proved to be extremely interesting in that much higher levels of oxidation were present in the tailings than any tested at INCO. Unique information with respect to the resistivity piezocone and its use in sulphide tailings characterization was indicated by the response in the unsaturated tailings where very definite trends were measurable. The work at the Fault Lake Tailings site was the first indication that the resistivity piezocone may be able to produce meaningful readings in less than fully saturated media.

I-1.2.3.3 Typical Data

Sounding - F01-9333 (Fault Lake)

The resistivity sounding for this location is shown on Figure I.5. The upper soil strata to a depth of approximately 4.6 m consists of a compact silty sand tailings, as indicated by the cone bearing and friction ratio. Interbedded layers of silty-sand and clayey-silt tailings and then native materials underlie this material; it was difficult to discern the break between the tailings and the native deposits if the latter were indeed intersected in the sounding. However, the much "calmer" response in all channels below a depth of about 28 m is indicative of a relatively homogenous deposit which is more likely due to a lacustrine clay than for highly layered tailings.

The native material, a clayey-silt, was considered to be encountered at a depth of approximately 28 m. The RCPTU met refusal at a depth of 41 m in what was considered to be a very dense gravely till material.
Sounding F02-9334 (Fault Lake)
Figure 1.6 shows a summary of sounding F02-9334. The tailings were found to extend to a depth of approximately 18.3-m. The tailings consist of interbedded layers of silty-sand and clayey-silt, which is in good agreement with F01-9333. A uniform silty clay native deposit underlies the tailings. The sounding met refusal at a depth of 22.75 m in very dense gravely till material.

I-1.3 CANLEX Project

I-1.3.1 General

As described in Chapter 5, the CANLEX project was divided into different phases, with each Phase extending for approximately one year with a specific set of objectives. The research for this thesis benefited from four phases of the project:

- Phase I - characterization work at Syncrude Canada Ltd.'s (SCL) Mildred Lake Settling Basin (MLSB), near Fort McMurray, Alberta.
- Phase II - characterization efforts on the Fraser River Delta of British Columbia.
- Phase III - static liquefaction triggering event at SCL's J-Pit.
- Phase IV - site characterization at Highland Valley Copper’s (HVC) tailings facility near Logan Lake, British Columbia.

Project Phases I, III and IV involved mine tailings whereas Phase II doubled as this research's local calibration test sites. This research utilized data from all four phases of the CANLEX project.

I-1.3.2 Syncrude Mine, Alberta

I-1.3.2.1 Site Description

The Syncrude Canada Ltd. mine, which is located in the Athabasca Oil Sand deposit approximately 40 km north of Fort McMurray in northern Alberta, started production in 1978. Syncrude processes approximately 150 to 160 million tonnes of oil sand per year. From mine startup to 1991, the Mildred Lake Settling Basin directly north of the Plantsite was the sole tailings storage area for the mine. Syncrude commissioned the Southwest Sand Storage (SWSS) Facility, located approximately 7 km southwest of the extraction plant, in 1991 to provide storage for coarse tailings. In 1998, approximately $3 \times 10^6$ m$^3$ of sand tailings were placed in cells along the toe areas of Mildred Lake and $30 \times 10^6$ m$^3$ of sand tailings were deposited on beaches at the SWSS Facility. A site location plan for the Syncrude Mine is shown on Figure I.7.
I-1.3.2.2 Research Program

The characterization work directly involving the Author consisted of regular (CPTU) and seismic piezocone (SCPTU) penetration tests and energy calibrated standard penetration testing (SPT) in rotary advanced boreholes as executed during Phase I of the CANLEX project. As indicated above, there were a number of field programs carried out at Syncrude over the CANLEX project duration. Also as noted, the Author’s thesis research included data from all of these programs which involved data from the Mildred Lake Settling Basin, SWSS and Syncrude’s J-Pit tailings area.

The initial fieldwork for Phase I was carried out at Mildred Lake between May 19 and May 22, 1994 inclusive in the Cell 24 Area shown on Figure I.8. A total of one CPTU and four SCPTU soundings were carried out to maximum depths of just over 39 metres. The locations of the soundings, denoted CNLX 9401 to CNLX 9405 inclusive, are shown on Figure I.9. Figure I.9 is a detailed plan for CANLEX's Phase I test site and includes the locations of all sampling and testing carried out to that date as part of the program. Two SPT boreholes were carried out, CNLX 9406 and CNLX 9407, and were located as shown on Figure I.9.

The CPTU/SCPTU work was carried out using the British Columbia Department of Transportation and Highways (BCMOTH) specialized in-situ testing vehicle that had a gross vehicle weight for the project of approximately 18 tonnes. The SPT work was carried out with the assistance of Elgin Explorations Ltd. who supplied a rubber tired Acker drill rig. Elgin also was used for pre-boring the majority of the CPTU/SCPTU sounding locations to the target depth of approximately 28 metres.

The J-Pit area discussed in the main text is shown on Figures I.10 and I.11. Figure I.11 includes the piezocone sounding labels for correspondence to the cross sections presented in the main text.

I-1.3.2.3 Typical Results

Sample results from J-Pit and SWSS are presented in the main text. The CANLEX references noted can be used for obtaining summary piezocone plots for any of the three areas on the Syncrude site. For the Mildred Lake site, as detailed in UBC-ISTG (1994), Figure I.12 presents a typical piezocone sounding from the Phase I site. Figure I.12 shows that the “target zone”, roughly from 27 m to 36 m, consists of a relatively uniform tailings sand with F of about 0.6% and not excessive variability in in-situ density. Figure I.13 shows a summary of the seismic wave data procured during the Phase I investigations. Very little variation was shown in this data, particularly in the “target” zone for the CANLEX Phase I characterization work.

I-1.3.3 Lower Mainland Sites

I-1.3.3.1 Sites Description

The CANLEX project utilized two Lower Mainland sites as its Phase II locations. These sites, the Kidd II Substation and Massey South, located as shown on Figure I.14, also served as local test calibration sites for the Author’s research. At both sites, the CANLEX target soil was...
located within the upper 20 to 30 metres of complex distributary channel sands and silty sands that underlies most of the Fraser River subaerial delta.

I-1.3.3.2 Research Program

As these sites also represented the research's test calibration locations, more than 50 resistivity and seismic piezocone soundings were completed in combination at the two sites. The Author also participated in obtaining the official CANLEX piezocone soundings at both sites which are located as shown on Figures I.15 and I.16 for the Massey South and Kidd II sites, respectively. Also shown on Figures I.15 and I.16 are the locations of the other in-situ tests, including the location of the frozen core for undisturbed sampling, carried out at each site for the CANLEX project.

I-1.3.4 Highland Valley Copper

The CANLEX project added a Phase IV site to its original program that was a mine tailings site. The Highland Valley Copper Mine (HVC), in British Columbia, provided the CANLEX project with a milled mine tailings versus the non-ground tailings at the Syncrude site. Figures I.17 and I.18 show the relative location of the piezocone soundings with the frozen core zones and other in-situ tests carried out at both the LL Dam and Trojan Dam, respectively, at HVC.

I-1.4 SCBC Project

I-1.4.1 General

As indicated in the main text, the SCBC project was the largest single contributor to the research database. Field trips to several mine sites were carried out in 1994, 1995 and 1996. In addition, the SCBC project provided the research funding for much of the tool development described in Chapter 4 of the thesis. The field trips and their duration is listed in Table I.2.

<table>
<thead>
<tr>
<th>MINE SITE</th>
<th>1994</th>
<th>1995</th>
<th>1996</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gibraltar</td>
<td>Aug 29 - Sept. 8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trail</td>
<td>Oct. 17 - 20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sullivan</td>
<td>Oct. 4 - 14</td>
<td>July 18 - 26</td>
<td>July 30 - Aug. 9</td>
</tr>
<tr>
<td>Endako</td>
<td>Aug. 15 - 22</td>
<td>July 3 - 12</td>
<td></td>
</tr>
</tbody>
</table>

The majority of the research work was completed utilizing the UBC-ISTG In-Situ Testing vehicle. Water samples and soil samples were collected to augment the piezocone information, and laboratory tests were conducted on these samples. Surface geophysical methods were also used at each of the mine sites. The basic statistics for the all the tests conducted as part of the SCBC program are listed in Table I.3.
Table I.3  Tests Statistics

<table>
<thead>
<tr>
<th>RCPTU DISSIPATION</th>
<th>CYCLIC</th>
<th>SEISMIC</th>
<th>IP</th>
<th>PORE WATER CONDUCTIVITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>113 tests = 1800 m</td>
<td>370 tests</td>
<td>3 tests</td>
<td>76 tests</td>
<td>31 tests</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>WATER SAMPLES</th>
<th>SOIL SAMPLES</th>
<th>EM 31 &amp; EM 34</th>
<th>KBAT (outflow)</th>
<th>SLUGTEST</th>
<th>PRESSUREMETER</th>
</tr>
</thead>
<tbody>
<tr>
<td>60 samples</td>
<td>2 samples</td>
<td>27 km</td>
<td>5 tests</td>
<td>5 tests</td>
<td>17 tests</td>
</tr>
</tbody>
</table>

(Gibraltar data not included)

The in-situ testing vehicle was used to collect the data for all the tests except soil sampling, slug-tests, and the EM 31 and EM 34 surface geophysics.

The reference list to the main body of the thesis lists the key references developed for each site program noted in Table I.2. Site location plans, test location plans and summary test information can be found in the appropriate reference for each site. These SCBC reports, developed by the Author, represent a good portion of the documented database for the research. Following are basic descriptions of each of the four sites and some indication of the nature of the research carried out in each case.

I-1.4.2 Sullivan Mine, British Columbia

I-1.4.2.1 Site Description

Cominco's Sullivan Mine is located in Kimberley, British Columbia. The mine is currently an underground operation that has operated on a near-continuous basis since the early 1900s. Present processing can be up to about 7,000 tonnes per day of primarily lead/zinc ore. Based on this average production rate and the estimated ore reserves, it is expected that the remaining mine life will be to December 31, 2001.

The Sullivan Concentrator is located on the southeast side of Kimberley, north of the town of Marysville. For most of the mine’s operating life, mill tailings have been hydraulically transported to an area immediately southeast of the concentrator for disposal and storage. Additional facilities were developed in 1978 for the storage of sludge waste from operation of the water treatment plant. The storage of gypsum tailings and circulation water from operation of the fertilizer plant has also been stored in the tailings area. These by-products from the fertilizer plant were produced and stored from about 1969 to 1987.

There are fourteen earthfill dykes that create six separate impoundments. The summary statistics for these six impoundments and fourteen dykes are summarized on Table I.4. These earthfill dykes have a combined length of approximately 10-km, with maximum heights varying from about 4 m to 30 m.
Table I.4 Summary of Impoundment Areas at Sullivan Mine

<table>
<thead>
<tr>
<th>IMPOUNDMENT AREA</th>
<th>EMBANKMENTS</th>
<th>APPROXIMATE EMBANKMENT LENGTH (m)</th>
<th>APPROXIMATE MAXIMUM HEIGHT (m)</th>
<th>STARTER Dyke CONSTRUCTED (year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 a) Iron Pond</td>
<td>Iron Dyke</td>
<td>1500</td>
<td>30</td>
<td>1975</td>
</tr>
<tr>
<td>1 b) Old Iron Pond</td>
<td>Southwest and Southeast Limbs</td>
<td>1700</td>
<td>11</td>
<td>-</td>
</tr>
<tr>
<td>2) Silica Ponds</td>
<td>No. 1 Silica Dyke</td>
<td>2000</td>
<td>17</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>No. 2 Silica Dyke</td>
<td>600</td>
<td>11</td>
<td>1975</td>
</tr>
<tr>
<td></td>
<td>No. 3 Silica Dyke</td>
<td>1500</td>
<td>12</td>
<td>1975</td>
</tr>
<tr>
<td>3) Gypsum Ponds</td>
<td>East Gypsum Dyke</td>
<td>650</td>
<td>17</td>
<td>1969</td>
</tr>
<tr>
<td></td>
<td>West Gypsum Dyke</td>
<td>650</td>
<td>23</td>
<td>1969</td>
</tr>
<tr>
<td></td>
<td>Northeast Dyke</td>
<td>120</td>
<td>10</td>
<td>1985</td>
</tr>
<tr>
<td>4) Calcine Pond</td>
<td>Calcine Dyke</td>
<td>510</td>
<td>50</td>
<td>1972</td>
</tr>
</tbody>
</table>

Typically the dykes consist of an initial earthfill starter section which were then raised incrementally over the years using the upstream method of construction. The Active Iron Pond presently the only operating impoundments at the Sullivan Mine. The other facilities have been decommissioned and are in various stages of reclamation in preparation for mine closure in December, 2001.

I-1.4.2.2 Research Program

The Sullivan Mine was utilized for all three years of the SCBC project and was the single largest contributor to the research database. In addition, as summarized in Chapter 9, the Sullivan Mine provided the research with a static liquefaction case history.

I-1.4.3 Gibraltar Mine, British Columbia

I-1.4.3.1 Site Description

Gibraltar Mine is an open pit copper-molybdenum mine which had been in operation since 1972. It is located near McLeese Lake in central British Columbia, approximately 50 km north of Williams Lake. Prior to shutdown in late 1998, the production rate was about 40,000 tonnes per day.

The mill tailings consist of a fine sand with about 50% fines passing the No. 200 sieve. The tailings are pumped in a slurry at 40% solids by weight to the tailings impoundment which is situated in a valley on the east fork of Cuisson Creek, about 7 km north of the plant site.

The tailings impoundment was commissioned in 1972 by construction of a 30 m high starter dam of compacted glacial till across the outlet of the creek valley. Subsequent dam raising has been performed using the centreline method of construction whereby tailings underflow from on-dam cyclones located across the crest of the dam was deposited directly onto the downstream slope of the dam. The cyclone underflow consists of relatively free draining fine sand with fines contents historically ranging between 7% and 13% based on Gibraltar's dry sieve gradation test procedures. Washing of tailings samples over a No. 200 sieve according to ASTM D-422 standard test procedure yields 3% to 7% higher fines content. No mechanical compaction is applied to the cycloned sand deposited on the slope of the dam. The cyclone overflow and "bleed" from the end
of the tailings line are discharged onto the tailings beach upstream of the dam centreline. Typically, the beach slopes down over a 2000 m length to the pond.

I-1.4.3.2 Research Program

The Gibraltar Mine was the first location investigated as part of the SCBC project. The tailings were highly amenable to the piezocone technology. The pore geochemistry was quite low in TDS and did not pose a problem for RES001 or the early version of RES002.

I-1.4.4 Endako Mine, British Columbia

I-1.4.4.1 Site Description

The Endako molybdenum mine is located approximately 100 miles west of the town of Prince George in north central B.C. Open pit mining operations began in 1965, and the tailings generated from the milling of the ore have been consigned to two separate disposal facilities, namely Ponds 1 and 2. When production started in 1965, the initial ore production rate was approximately 10,000 tpd. Production was increased to about 26,000 tpd in 1970, and subsequently reduced in the latter half of that year. Mining and milling operations were curtailed in 1982 due to low metal prices. Production was resumed in 1986 at a rate of approximately 10,000 tpd. Mill throughput was increased to approximately 26,000 tpd in 1989. Current production is on the order of 30,000 tpd.

The tailings are deposited in the ponds via a tailings pipe line that is located around the perimeter of the dam crests. Small 1 m to 2 m high tailings berms are build along the inside of the pipeline. The tailings used to construct the berms are built along the inside of the pipeline. The tailings used to construct the berms are obtained from the adjacent beach by pushing with bulldozers. Spigot pipes that exit from the bottom of the main perimeter pipeline are draped over the small berms, and the tailings are deposited on the adjacent beach. This process is repeated when the beach is raised to the crest of the small berms, and, consequently, the dam is raised in an upstream manner.

I-1.4.4.2 Research Program

The Endako Mine provided the second largest contribution of data to the SCBC program. The combination of upstream constructed tailings, relatively high water tables in the tailings and a reasonable amount of geochemical and geotechnical reference data from conventional testing (by others) made this site an important one for the research. Two extended field programs were completed at Endako in 1995 and 1996.

I-1.4.5 Trail Smelter, British Columbia

The Trail Smelter site is actually a large industrial development over several km². The area of interest for the SCBC program is known as the Duncan Flats area, an area of previous arsenic waste storage on native granular soils. The research program specifically concentrated on the Duncan Flats area and the area directly below the Flats adjacent to Topping (Stoney) Creek. In total, the program included nine resistivity piezocone soundings, just over 5000 metres of
electromagnetic surface geophysics and surface water conductivity readings along Topping Creek.

The work at the Trail Smelter provided the first indication that the bulk resistivity signature from the resistivity piezocone could be related to the surface electromagnetic signal from the Geonics™ EM-31. The bulk resistivity from both the resistivity piezocone and the EM-31 showed an ability to detect the presence of elevated arsenic in both tension saturated soils and below the groundwater table.
Figure I.4  RCPTU Sounding - I19-9323 (Base of Pistol Dam)
Figure I.5  RCPTU Sounding - F01-9333 (Centre of Fault Lake Tailings Area)
Figure I.7  Syncrude Canada Ltd. - Site Location Plan
Figure 1.8  Mildred Lake Settling Basin - Phase I and Phase III - CANLEX Sites (adapted from Robertson et al, 1993).
Figure I.9  Detailed Location Plan at CANLEX Phase I Site (adapted from UBC-ISTG, 1994)
Figure I.10  Configuration of J-Pit Phase III CANLEX Site
Figure 6 CANLEX-SYNCRUDE 1994

Operators: HPD-RSJ-TJB-FM  CPT Date: 05/22/94 11:48  File Name: CNLX9405.EDT
Location: MILDRED LK CELL24  Cone Used: HOG2U2wSEISMOM  Comments: UBC IN-SITU with BCMOTH

FRICITON RATIO
Rf=Fs/Qtx100 (K)

BEHIND TIP PORE PRES
U2 (m. of water)

COE TIP STRESS
Qt (bar)

SLEEVE FRICTION
Fs (bar)

TEMPERATURE
(DEGREES CELCIUS)

DEPTIh (meters)

Depth Increment: .025m  GWT-22m  Refusal Depth: 38.00m

Figure 1.12 Piezocone Sounding - CNLX 9405 (Cell 24 Phase I CANLEX)
Figure I.13  Shear Wave Velocity Data (Cell 24 CANLEX Phase I)
Figure I.14  CANLEX Phase II Lower Mainland, British Columbia - General Location Plan
Figure 1.15: CANLEX Phase II Massey South Site Detailed Site Plan
Figure I.16  CANLEX Phase II Kid #2 Detailed Site Plan
DEEP SPT2 (14.7m)
DEEP SCPTU4 (56.6m)
DEEP GEO2 (23m)
LLSBPM2 & LLSBPM3
AND 15 sq. cm CPT
LDS
10 m DIAMETER
GROUND FREEZING
TO WATERLINE (-22 m)

GEO: GEOPHYSICAL LOGGING
SPT: STANDARD PENETRATION TEST
LDS: LARGE DIAMETER SAMPLING

Figure I.17  LL Dam Detailed Location Plan (adapted from Biggar and Robertson, 1996)
Figure I.18 Trojan Dam Detailed Location Plan (adapted from Biggar and Robertson, 1996)
APPENDIX II

Surface Electromagnetics in Relation to RCPTU

Fortran Computer Programs
APPENDIX II
SURFACE ELECTROMAGNETICS IN RELATION TO
RCPTU-COMPUTER PROGRAM

PICK.FOR program - used to isolate and calculate the
resistivity at specified depths.

Program PICK

c Program used to read a series of RCPTU files and
c pick the resistivity value at certain depths to
c write those to a dedicated file in order to plot
c using a plotting package. The result is a single
c file with all the resistivity data for a certain
c depth at certain locations.
c
VARIABLES
c
--
c Initialize variables
c --
    Character*12 infile,outfile,label
    Integer ave,num,count
    Real depth(11),res(11),north,east,int,int2,step,max,
    + averes,rdiff,resdiff
c --
c Input parameters - depth,averaging,etc.
c --
    open (4,file='readfile.dat',status='old')
    read (4,480) num,int,step,ave
    count=1
    int2=int
5    read (4,490) north,east,infile,outfile,label,max
    int=int2
    write (*,'(A12,2x,f7.3)') outfile,(max-(5*step))
c --
c Open the appropriate RCPTU file to read resistivity data
c and correct file for writing data too.
c --
    Open (5, file=infile, status='old')
    Open (6, file=outfile, status='unknown')
c --
c Write header to file for *.RES
c --
    Write (6,'(2f8.3)') max,int

423
c --
c Skip header in RCPTU files.
c --
   Do 10,1=1,2
      Read (5,‘(50X)’)
   10 Continue
c --
c Read in next line of data and check if needed
c Adjust parameters
c --
20   I=1
    Read (5,500) depth(I),res(I)
    If (depth(I).GE.(int-((ave*step)+0.01))) then
      Do 30,I=2,(1+(2*ave))
      If (depth(I-1).GT.(max-(5*step))) goto 50
      Read (5,500) depth(I),res(I)
    30 Continue
c --
c Calculate average resistivity
c --
   averes=0.0
   resdiff=0.0
   J=0
   Do 40,1=1,(1+(2*ave))
      If (res(I).LT.0.01) then
         J=J+1
         goto 45
      Endif
      averes=averes+res(I)
   45 If (I.GE.2) then
      rdiff=((res(I)-res(I-1))**(2)**0.5
      If (rdiff.GT.resdiff) then
         resdiff=rdiff
      endif
   endif
   40 Continue
   if (averes.LT.0.01) goto 50
   averes=averes/((1+(2*ave))-J)
50   write (6,520) north,east,averes,int,label,resdiff
c --
c adjust parameters depth interval
c --
   int=int+int2
Endif
   If (depth(I).LT.(max-(5*step))) goto 20
c --
c Check to determine if all files considered
c --
  If (count.lt.num) then
    count=count+1
    goto 5
  Endif

c --
c Formats
c --
480  Format (I3,2f7.3,I3)
490  Format (2f8.1x,A12,1x,A12,1x,A10,f7.3)
500  Format (lf7.4,58x,lf6.2)
520  Format (lf11.3,lf10.3,2f7.2,lx,A10,f7.2)
c --
c END PROGRAM
c --
  Stop
  End
READ FILE, DAT file - used to give program data about files to be read

<table>
<thead>
<tr>
<th>No. of files, spacing, interval, points to average</th>
<th>N, E, Input, Output, Label, Maximum Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>18 0.500 0.025 1</td>
<td>-13914.5 15101.0 SV19406A.COR SV1-9406.RES &quot;SV1-9406&quot; 6.450</td>
</tr>
<tr>
<td></td>
<td>-14979.1 14902.2 SV1-9424.COR SV1-9424.RES &quot;SV1-9424&quot; 10.425</td>
</tr>
<tr>
<td></td>
<td>-14834.8 15342.9 SV1-9425.COR SV1-9425.RES &quot;SV1-9425&quot; 10.400</td>
</tr>
<tr>
<td></td>
<td>-13845.4 15726.7 SV1-9429.COR SV1-9429.RES &quot;SV1-9429&quot; 2.575</td>
</tr>
<tr>
<td></td>
<td>-13819.5 15542.4 SV1A9430.COR SV1-9430.RES &quot;SV1-9430&quot; 6.075</td>
</tr>
<tr>
<td></td>
<td>-12799.5 15705.5 SV1A9402.COR SV1A9402.RES &quot;SV1A9402&quot; 14.475</td>
</tr>
<tr>
<td></td>
<td>-13009.7 15607.1 SV1A9403.COR SV1A9403.RES &quot;SV1A9403&quot; 15.150</td>
</tr>
<tr>
<td></td>
<td>-13163.0 15491.3 SV1A9404.COR SV1A9404.RES &quot;SV1A9404&quot; 15.450</td>
</tr>
<tr>
<td></td>
<td>-12695.9 15404.6 SV1A9420.COR SV1A9420.RES &quot;SV1A9420&quot; 5.000</td>
</tr>
<tr>
<td></td>
<td>-12877.9 15336.5 SV1A9421.COR SV1A9421.RES &quot;SV1A9421&quot; 8.550</td>
</tr>
<tr>
<td></td>
<td>-13046.4 15274.2 SV1A9422.COR SV1A9422.RES &quot;SV1A9422&quot; 10.400</td>
</tr>
<tr>
<td></td>
<td>-13630.9 15812.7 SV1A9428.COR SV1A9428.RES &quot;SV1A9428&quot; 5.025</td>
</tr>
<tr>
<td></td>
<td>-12164.0 15911.8 SV2-9401.COR SV2-9401.RES &quot;SV2-9401&quot; 13.775</td>
</tr>
<tr>
<td></td>
<td>-12178.1 16080.1 SV2-9405.COR SV2-9405.RES &quot;SV2-9405&quot; 12.900</td>
</tr>
<tr>
<td></td>
<td>-11986.7 15779.1 SV2-9417.COR SV2-9417.RES &quot;SV2-9417&quot; 15.300</td>
</tr>
<tr>
<td></td>
<td>-12119.3 15706.0 SV2-9418.COR SV2-9418.RES &quot;SV2-9418&quot; 10.075</td>
</tr>
<tr>
<td></td>
<td>-12263.4 16241.0 SV2-9419.COR SV2-9419.RES &quot;SV2-9419&quot; 1.375</td>
</tr>
<tr>
<td></td>
<td>-12106.2 15822.5 SV2-9432.COR SV2-9432.RES &quot;SV2-9432&quot; 13.725</td>
</tr>
</tbody>
</table>
LEVEL.FOR code - program used to create depth files with conductivity data.

Program LEVEL

c c Program used to take output from PICK.FOR 
c and sort it to put into files of appropriate 
c depth for surfer.
c
VARIABLES

c --
c Initialize variables

Character*12 infile(50),outfile(50),label
Integer num,numfile,ctl,ct2
Real depth,res,north,east,max,resdiff,int,cond1,cond2
ct1=1
c2=1
c --
c Read in and assign output depth files

open (3,file='depthres.dat',status='old')
read (3,'(13)') numfile
Do 10,I=1,numfile
Read(3/(A12)') outfile(I)
10 Continue
Close (3)
c --
c Create Empty depth files

Do 20,I=1,numfile
Open (6 ,file=outfile(I),status='unknown')
Close (6)
20 Continue
c --
c Read in data files created by PICK.FOR [* RES]

open (4,file='inputres.dat',status='old')
read (4,'(i3)') num
Do 30,I=1,num
read (4,'(A12)') infile(I)
30 Continue
c --
c Open a *.RES file and set depth max

Open (5,file=infile(ctl),status='old')
Read (5,'(2f8.3)') max,int
c --
c Read in data lines and sort
  
c --
50  Read (5,520) north,east,res,depth,label,resdiff
  
c --
c Convert resistivity to conductivity (microS/cm)
c --
  cond = 0.0
  if (res.gt.0.0) then
    cond1 = ( 10000 / res )
    cond2 = ( 1000 / res )
  Endif
  
c --
c Open the correct depth file to place readings into
c --
  Open (6, file=outfile(ct2), status='append')
  Write (6,530) north,east,cond1,cond2,depth,label,resdiff
  Close (6)
  
c --
c Check to see if data file is exceeded
  
c --
  if (((ct2*int)+(1.5*int)).gt.(max)) then
    ct1=ct1+1
    if (ct1.gt.num) goto 100
    ct2=1
    close (5)
    goto 40
  Endif
  
c --
c limit depth search to 10 m
  
c --
  if ((ct2*int)+int.gt.10) then
    ct1=ct1+1
    if (ct1.gt.num) goto 100
    ct2=1
    close (5)
    goto 40
  Endif
  
c --
c adjust parameters depth interval
  
c --
ct2=ct2+1
  goto 50
  
c --
c Formats
  
c --
520  Format (1f11.3,1f10.3,2f7.2,1x,A10,f7.2)
530 Format (1f11.3,1f10.3,3f10.2,1x,A10,f7.2)
c--
c END PROGRAM

c--
100 Stop
    End
DEPTHRES.DAT file - used to run LEVEL.FOR

20 | number of files to be read
res050.dat | Data file names to be created
res100.dat
res150.dat
res200.dat
res250.dat
res300.dat
res350.dat
res400.dat
res450.dat
res500.dat
res550.dat
res600.dat
res650.dat
res700.dat
res750.dat
res800.dat
res850.dat
res900.dat
res950.dat
res1000.dat

INPUTRES.DAT file - used to run LEVEL.FOR

18 | Number of files to be read
SV1-9406.RES | Files to be read for resistivity data
SV1-9424.RES
SV1-9425.RES
SV1-9429.RES
SV1-9430.RES
SV1A9402.RES
SV1A9403.RES
SV1A9404.RES
SV1A9420.RES
SV1A9421.RES
SV1A9422.RES
SV1A9428.RES
SV2-9401.RES
SV2-9405.RES
SV2-9417.RES
SV2-9418.RES
SV2-9419.RES
SV2-9432.RES
SV1-9406.RES - Output from PICK.FOR
| Maximum Depth, Interval |
| 6.450 | 0.500 |
| -13914.500 | 15101.000 | 7.45 | 0.50 "SV1-9406" | 7.45 |
| N, E, Resistivity, Depth, Label, Max Resitivity Diff |
| -13914.500 | 15101.000 | 0.00 | 1.00 "SV1-9406" | 0.00 |
| -13914.500 | 15101.000 | 53.76 | 1.50 "SV1-9406" | 6.25 |
| -13914.500 | 15101.000 | 71.08 | 2.00 "SV1-9406" | 10.05 |
| -13914.500 | 15101.000 | 13.35 | 2.50 "SV1-9406" | 2.59 |
| -13914.500 | 15101.000 | 34.63 | 3.00 "SV1-9406" | 3.23 |
| -13914.500 | 15101.000 | 39.28 | 3.50 "SV1-9406" | 5.08 |
| -13914.500 | 15101.000 | 37.75 | 4.00 "SV1-9406" | 2.66 |
| -13914.500 | 15101.000 | 28.55 | 4.50 "SV1-9406" | 3.94 |
| -13914.500 | 15101.000 | 15.27 | 5.00 "SV1-9406" | 1.40 |
| -13914.500 | 15101.000 | 22.09 | 5.50 "SV1-9406" | 2.31 |
| -13914.500 | 15101.000 | 0.00 | 6.00 "SV1-9406" | 0.00 |

RES050.DAT - Output from LABEL.FOR (Called CON050.DAT on disk)

| N, E, Conductivity microS/cm, mS/m, Depth, Label, Max Difference in Resistivity |
| -13914.500 | 15101.000 | 1342.28 | 134.23 | 0.50 "SV1-9406" | 7.45 |
| -14979.100 | 14902.200 | 21.30 | 2.13 | 0.50 "SV1-9424" | 32.10 |
| -14834.800 | 15342.900 | 50.55 | 5.06 | 0.50 "SV1-9425" | 223.89 |
| -13845.400 | 15726.700 | 306.84 | 30.68 | 0.50 "SV1-9429" | 36.90 |
| -13819.500 | 15542.400 | 174.70 | 17.47 | 0.50 "SV1-9430" | 81.54 |
| -12799.500 | 15705.500 | 4201.68 | 420.17 | 0.50 "SV1A9402" | 2.63 |
| -13009.700 | 15607.100 | 106.41 | 10.64 | 0.50 "SV1A9403" | 136.71 |
| -13163.000 | 15491.300 | 4098.36 | 409.84 | 0.50 "SV1A9404" | 1.76 |
| -12695.900 | 15404.600 | 341.18 | 34.12 | 0.50 "SV1A9420" | 30.84 |
| -12877.900 | 15336.500 | 151.49 | 15.15 | 0.50 "SV1A9421" | 29.80 |
| -13046.400 | 15274.200 | 740.19 | 74.02 | 0.50 "SV1A9422" | 3.02 |
| -13630.900 | 15812.700 | 946.07 | 94.61 | 0.50 "SV1A9428" | 3.02 |
| -12164.000 | 15911.800 | 365.10 | 36.51 | 0.50 "SV2-9401" | 21.34 |
| -12178.100 | 16080.100 | 6060.61 | 606.06 | 0.50 "SV2-9405" | 0.27 |
| -11986.700 | 15779.100 | 21.67 | 2.17 | 0.50 "SV2-9417" | 0.11 |
| -12119.300 | 15706.000 | 20.84 | 2.08 | 0.50 "SV2-9418" | 33.18 |
| -12263.400 | 16241.000 | 21.72 | 2.17 | 0.50 "SV2-9419" | 1.17 |
| -12106.200 | 15822.500 | 2487.56 | 248.76 | 0.50 "SV2-9432" | 1.66 |