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Canadian Liquefaction Experiment
CANLEX
1993-1997

Characterization of Sand for Dynamic and Static Liquefaction

"The CANLEX Project was awarded the Association of Professional Engineers, Geologists and Geophysicists of Alberta (APEGGA) Project Achievement Award in 1998. This award recognizes a project which demonstrates engineering, geological or geophysical skills and represents a substantial contribution to a technical progress and the betterment of society".

Introduction

The Canadian geotechnical engineering community has completed a major collaborative research project entitled the Canadian Liquefaction Experiment (CANLEX). The phenomenon of soil liquefaction can occur in saturated sandy soils and is characterized by a large loss of strength or stiffness resulting in substantial deformation. In many areas of Canada, large structures are constructed on or comprised of sandy soils. Examples of such structures are tailings impoundments developed by the mining industry and some major earth dams used for hydro-electricity.

The behaviour of loose sandy soils can be difficult to predict, but can have a significant financial impact on these types of engineering structures. The characterization of loose sandy soils is an area of uncertainty in geotechnical engineering. Unlike clay soils, it is almost impossible to obtain undisturbed samples of loose sandy soils, especially at depth, using conventional methods. Hence, in-situ testing techniques have become standard practice for sand characterization and the evaluation of liquefaction potential.

The objectives of the CANLEX Project were:

- develop test sites to study sand characterization
- develop and evaluate undisturbed sampling techniques
- calibrate and evaluate in-situ testing techniques
- obtain an improved understanding of the phenomenon of soil liquefaction
Project Participants

CANLEX was a collaborative project with participation from industry, engineering consultants and universities. The participants were, as follows:

Industry:
- Syncrude Canada Ltd.
- Suncor Inc.
- Hydro Quebec
- Kennecott Corporation, USA
- Highland Valley Copper

Universities
- University of Alberta
- University of British Columbia
- Université de Laval
- Carleton University

Engineering Consultants:
- AGRA Earth and Environmental Ltd.
- ERA Consultants Ltd.
- Golder Associates Ltd.
- Klohn-Crippen Consultants Ltd.
- Thuber Engineering Ltd.

Others
- Canadian Geological Survey (CGS)
- B.C. Min. of Highways (BC MOTH)
- ConeTec Investigations Ltd.
- Hughes In-situ Engineering Inc.

Summary of Progress

The CANLEX Project was carried out from February, 1993 to December, 1997. The Project was divided into Phases with each Phase representing essentially a new site and/or research objective. A total of five (5) project Phases were carried out over the approximately 5 year period. Each project Phase was subdivided into a series of activities, with each activity assigned an activity leader and group of participants.

Three main soil types were tested, each located in different regions. Phases I and III were carried out at the Syncrude Canada Ltd. site located north of Fort McMurray in Alberta. The Syncrude soil was a tailings sand resulting from oil extraction from the oil sand deposits. This was a natural sand which had been processed to remove the oil and then hydraulically placed in impoundments.

Phase I was located in the existing Mildred Lake Settling Basin, whereas Phase III was located in freshly deposited tailings placed in an old overburden pit, referred to as J-Pit. Phase II was carried out in the Fraser River delta region near Vancouver, B.C. Two sites were selected for Phase II and consisted of natural deposits of Fraser River sand. Phase IV was carried out at the Highland Valley Copper Mine in central British Columbia, south of Kamloops, B.C. Two sites were selected for Phase IV and consisted of tail-
Undisturbed sampling using large diameter (200mm) CRREI sampler in sand frozen using liquid nitrogen.

ings sand from the hard rock mining operations of Highland Valley Copper. One site was located in an older tailings storage facility referred to as Highmont Dam. The other site was located in a currently used tailings storage facility referred to as LL Dam.

Phase III included the construction of a full scale embankment in an effort to simulate a flow liquefaction slide. Phase V was located at the site of the Phase III embankment at the Syncrude site in J-Pit and included several controlled blasting experiments in an effort to simulate earthquake loading within the loose saturated tailings sand deposit.

Each Phase (except Phase V) involved the following basic activities:

- site selection to determine the preferred site location.
- site characterization to perform drilling and in-situ testing, which included standard penetration testing (SPT), cone penetration testing (CPT), seismic CPT, self-boring pressuremeter testing (SBPMT) and geophysical logging.
- sampling, which included ground freezing to obtain undisturbed samples, large diameter piston sampling and conventional sampling using a fixed piston thin walled sampler.
- laboratory testing, which included testing on reconstituted samples as well as undisturbed samples. Laboratory testing was carried out to evaluate the response of the soil to both monotonic and cyclic loading.

**Summary of Results**

In general, all of the major objectives of the project have been achieved. The following is a summary of the major achievements:

- a consistent set of definitions for liquefaction phenomena was developed and accepted by the participants.
- six (6) test sites were located and fully characterized; each site has unique features related to grain characteristics, mineralogy, age, depth, density and variability.
- a new technique was developed and evaluated to obtain high quality undisturbed samples using ground freezing techniques.
- the application of the Laval large diameter piston sampler was evaluated.
- the application of high quality conventional fixed piston sampling was evaluated.
- improvements were made to undisturbed sample handling procedures.
- in-situ testing techniques were evaluated and calibrated against response from undisturbed samples.
- a thaw consolidation procedure was developed and evaluated for frozen samples.
• geophysical logging methods were evaluated, including a new radio-isotope CPT.
• the importance of consolidating samples in the laboratory under in-situ stresses (i.e. Ko consolidation) was identified and evaluated.
• the importance of direction of loading on the response of sand was identified and quantified.
• the importance of strain compatible analyses to evaluate ground deformations was identified.
• improvements were made in the understanding of the response of sand to different types of loading.
• improvements were made in the understanding of the density and variability of sand deposits.

The following are the non technical achievements:
• Improved understanding of the safety of structures involving sand deposits.
• Enhanced communication between industrial partners.
• Enhanced knowledge and reputation of consultants.
• Immediate technology transfer.
• Optimization of national geotechnical resources.

Collaboration
The CANLEX Project has been the largest collaborative research project within the Canadian geotechnical community. In general, the collaboration has been excellent. Expertise exists in all segments of the project (i.e. industry, consultants and universities) and participants have shared ideas and comments freely and openly.

Summary
The Canadian geotechnical engineering community has completed a major collaborative research project entitled the Canadian Liquefaction Experiment (CANLEX) with participation from industry, engineering consultants and universities. The project involved a total expenditure of $1,690,000 with approximately $2,246,000 of additional in-kind contributions. The CANLEX project has achieved all of its major objectives.

The results of the CANLEX Project have been published in a series of 70 internal reports, 5 newsletters, 31 conference papers, 5 news articles, 10 journal publications and 13 graduate student theses. Further journal and conference papers are planned as well as some remaining graduate student theses. Further research work has been identified and a future workshop as well as a possible international conference are planned.
Phases I and III Data Review Report
(Mildred Lake and J-pit Sites, Syncrude Canada Ltd.)

Volume II -- Appendices

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October 15, 1997
EXECUTIVE SUMMARY

This volume of the Phases I and III Data Review Report contains copies of the raw data from in-situ testing and laboratory testing associated with both the Phase I (Mildred Lake) and Phase III (J-pit) sites at Syncrude Canada Ltd. These data were generated by various participants in the CANLEX Project, as noted on the cover page for each appendix. The in-situ testing for each site is divided into SPT results, CPT results, shear wave velocity results from seismic CPTs, geophysical logging results and pressuremeter testing results. The laboratory testing for each site is divided according to the laboratories that conducted the testing (University of British Columbia, Université Laval, University of Alberta, and C-CORE) and consists of triaxial compression and extension tests, simple shear tests and cyclic simple shear (or cyclic triaxial) tests. For detailed descriptions regarding any of the in-situ tests or laboratory tests, the reader is referred to the original technical reports by the individual CANLEX participants (summarized in the list of references at the end of this volume of this report). Also included in this volume is an appendix containing copies of recent papers by Roy et al. (1996) and Hughes et al. (1997), which include the application of computer aided modelling (CAM) techniques to pressuremeter test results from the Phase I and/or Phase III sites. The final appendix in this volume contains copies of five papers, concerning the Phase III full scale liquefaction field test, that were presented at the 1996 Canadian Geotechnical Conference in St. John's, Newfoundland.
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APPENDIX A

Phase I SPT Results
(Campanella, 1994)
Table 2 - SPT Energy Calibration Data from CNLX 9406

<table>
<thead>
<tr>
<th>Mean Depth (m)</th>
<th>Raw SPT-N Blow Count (blows per last 300 mm)</th>
<th>% Theoretical Average Energy Using F² Method</th>
<th>% Theoretical Average Energy Using FV Method</th>
<th>(N)₆₀ Using FV Method for adjustment</th>
<th>Standard Deviation using F² Method</th>
<th>Standard Deviation using FV Method</th>
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Hammer No. 1, ♠Hammer No. 2

Table 3 - SPT Energy Calibration Data from CNLX 9407

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<th>Mean Depth (m)</th>
<th>Raw SPT-N Blow Count (blows per last 300 mm)</th>
<th>% Theoretical Average Energy Using F² Method</th>
<th>% Theoretical Average Energy Using FV Method</th>
<th>(N)₆₀ Using FV Method for adjustment</th>
<th>Standard Deviation using F² Method</th>
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Hammer No. 1, ♥Hammer No. 2

The results from both analytical methods, F² and FV, indicate 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. The N60 values tabulated in Tables 2 and 3 represent the 60% energy level recommended by many researchers for earthquake liquefaction analysis and design based on existing data bases of field performance (Seed, 1987 and Kayan et al, 1992). The depth of these tests and the resulting stress levels require special consideration as to the actual energy that should be adopted for comparative purposes to a database that is largely established at stress levels roughly 1/3 to 1/2 or less of those encountered here. Nonetheless, the results summarized in Tables 1 and 2 indicate that relatively constant energies were experienced over two different days with different hammers.
APPENDIX B

Phase I CPT Results
(Campanella, 1994)
Fig. 2 CANLEX-SYNCRUDE 1994

Operators: MPD-TJB-RM
Location: MILDRED LK CELL24
Cone Used: UBC9U2U3wACCEL
Comments: U8C IN-SITU with BCMOTH

CPT Date: 05/19/94 14:16
File Name: CNLX9401.EDT

Friction Ratio
Rf=Fs/Qtx100 (%)

Behind Tip Pore Pressure
U2 (m. of water)

Cone Tip Stress
Qt (bar)

Inclination (Degrees)

Temperature (Degrees Celsius)

Depth Increment: .025m
GWT-22m
Refusal Depth: 20.38m
Fig. 3 Canelx-Syncrude 1994

Operators: MPD-RSJ-TJB-RM
Location: Mildred Lk Cell24
File Name: CNLX9402.EDT
CPT Date: 05/20/94 15:04
Cone Used: UBC9U2U3wACCEL
Comments: UBC IN-SITU with BCMOTH

Friction Ratio
Rf-Fs/Qt x 100 (%)

Behind Tip Pore Pres
U2 (m. of water)

Cone Tip Stress
Qt (bar)

Behind Fs Sleeve PP
U3 (m. of water)

Temperature
(Degrees Celsius)

Depth Increment: .025m
GWT=22m
Refusal Depth: 38.78m
Fig. 4 CANLEX-SYNCRUD 1994

Operators: MPD-RSJ-TJB-RM
Location: MILDRED LK CELL24
Cone Used: UBC9U2U3wACCEL
Comments: UBC IN-SITU with BCMOTH

FRICITION RATIO
\( R_f = \frac{F_f}{\gamma d_t} \times 100 \%
\)

BEHIND TIP PORE PRES
\( U_2 \) (m. of water)

CONER TIP STRESS
\( q_t \) (bar)

BEHIND Fs SLEEVES PP
\( U_3 \) (m. of water)

TEMPERATURE
(DEGREES CELSIUS)

Depth Increment: .025m
GWT=22m
Refusal Depth: 39.28m
Fig. 5 CANLEX-SYMCRUDE 1994

Operators: MPD-TJB-AM
CPT Date: 05/21/94 15:45
Location: MILDRED LK CELL24
Cone Used: HOG2U2wSEISMOD
Comments: UBC IN-SITU with BCMOTH

Friction Ratio
\( R_f = \frac{F_s}{q_{xt}} \times 100 \) (%)

Behind Tip Pore Pres
\( U_2 \) (m. of water)

Cone Tip Stress
\( q_t \) (bar)

Sleeve Friction
\( F_s \) (bar)

Temperature
Degrees Celsius

DEPTH (meters)

Depth Increment: .025m
GWT=22m
Refusal Depth: 36.50m
APPENDIX C

Phase I Shear Wave Velocity Results
(Campanella, 1994)
### File Name: CNLX9402 May20/94

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Fig. 7 - CANLEX PHASE I - Cross-Over Shear Wave Velocity Profiles
APPENDIX D

Phase I Geophysical Results
(Küpper, 1994)
HBT AGRA Limited
Engineering & Environmental Services

CANLEX - PHASE I SITE
CENTURY GEOPHYSICAL CORP. LOGS
BOREHOLE CORE-2
TOOL 9036

SCALE SHOWN DATE JAN/94 MADE CHKO AGK JOE EG-07680 FIGURE 4.4
CANLEX - PHASE 1 SITE
GEOPHYSICAL LOG
BOREHOLE CORE 1

Void Ratio, (e)

0.0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8

0

4

8

12

16

20

24

28

32

36

40

44

48

Depth (metres)

--- CORE 1 (Log 3) --- CORE 1 (Log 4) --- CORE 1 (Log 5)

FIGURE 4.13
CANLEX - PHASE 1 SITE
GEOPHYSICAL LOG
BOREHOLE CORE 2

FIGURE 4.17
FIGURE 4.25
APPENDIX E

Phase I Pressuremeter Results
(Hughes, 1994)
a) Non-selfbored pressuremeter tests
Table 1. Preliminary results of the analysis of the data

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Notes on Table 1

(1) The critical state friction angle has been taken as 34 degrees.
(2) The \( G_{\text{sec}} \) is the secant modulus from the initial expansion until the sand starts to deform plastically.
(3) \( P_{10} \) is the total pressure at 10% strain.
(4) \( G_{\text{max}} \) is the slope of the load-unload loops.
(5) The Poisson's ratio required in the Carter analysis has been taken as 0.25.
(6) In view of the low water table, there is some uncertainty in the effect this has on the analysis of the data. In the above analysis it has been assumed that the fluid pressure from the drilling mud will be that in the sand near the pressuremeter.
Pressuremeter data  × Canlex Project Syncrude

Hole No. 1  Depth 65

20 November 1993

Shear Modulus (kPa)  138628

Radial displacement / Radius (%)
Analysis of Data * Canlex Project Syncrude

Hole No. pm1    Depth 65        20 November 1993

Sand model

- Initial shear modulus: 3000 kPa
- Friction angle: 37°
- Critical state friction angle: 34°
- Effective lateral stress: 370 kPa
- Static water pressure: 200 kPa

Pressure (kPa)

0 750 1500 2250 3000

Radial displacement %

0 3.75 7.5 11.25 15

canicom Shift .1
PRESSUREMETER DATA * Canlex Project Syncrude

Hole Number pm1  Depth 65  20 November 1993

Carter Sand model

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Static water pressure (kPa)

Radial displacement %

Hughes
Pressuremeter data * Canlex Project Syncrude

Hole No. pm1 Depth 70 20 November 199

Shear Modulus (kPa) 213341

Radial displacement / Radius (%)

HUGHES
Analysis of Data * Canlex Project Syncrude

Hole No. pm1 Depth 70 20 November 1994

Sand model

- Initial shear modulus
- Friction angle
- Critical state friction angle
- Effective lateral stress
- Static water pressure

Radial displacement %

50000
35
34
400
200

11.25
7.5
3.75
Radial displacement %

can2 Shift 0

Hughes
PRESSUREMETER DATA * Canlex Project Syncrude
Hole Number pm1 Depth 70 20 November 1994
Carter Sand model

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<td>Static water pressure (kPa)</td>
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Radial displacement %

Hughes
Pressuremeter data  x  Canlex Project Syncrude

Hole No. pm1   Depth 98

20 November, 1993

Shear Modulus (kPa)  123333

Radial displacement / Radius (%)
**Analysis of Data * Canlex Project Syncrude**

Hole No. pm1  Depth 98  20 November 1993

### Sand model

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![Graph](image_url)
PRESSUREMETER DATA * Canlex Project Syncrude

Hole Number pm1  Depth 98  20 November 1993

Carter Sand model

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Radial displacement %
Pressuremeter data

* Canlex Project Syncrude

Hole No. pm1 Depth 100

20 November 1993

Shear Modulus (kPa) 239814

Radial displacement / Radius (%)
Hole No. pm1
Depth 100
20 November 1993

Analysis of Data x Canlex Project Syncrude

Initial shear modulus
Friction angle
Critical state friction angle
Effective lateral stress
Static water pressure

Sand model

11.25
7.5
3.75
0

radial displacement %

15
can3 shift 1

HUGHES
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Pressuremeter data  *  Canlex Project Syncrude

20 November 1993

Hole No. pm1  Depth 105

Shear Modulus (kPa)

107449

Radial displacement / Radius (%)
Shear Modulus (kPa)

Radial displacement / Radius (%)

Pressure (kPa)

0.750

2250

Calibration Tube Test

Pressurimeter data X Canjex Project Syncrude

Hughes
Pressuremeter data  *  Canlex Project Syncrude

Calibration Air Test

Graph showing shear modulus (kPa) vs. radial displacement / radius (%). The graph has a linear trend with a point at 150 kPa and a percentage value along the x-axis.
b) Self-bored pressuremeter tests
Figure 1. Pressuremeter pressure / expansion curves of tests at 35, 35.8 and 36.6 m
Figure 2  Calibration test - Pressuremeter expansion inside a steel cylinder

Figure 3  Calibration test - Pressuremeter expansion in air
Average Field Data

Figure 4  Log/Log plot of effective pressure / strain

Figure 5  Dilation angle and friction angle from Robertson and Hughes (1986)
Figure 6 Predicted pressure/expansion curve using:
Effective lateral stress \((\sigma_b') = 200\) kPa, i.e. \(k_o = 0.4\)
Friction angle \(\phi=\phi_{ev}\) of 34 degrees

Figure 7 Predicted pressure/expansion curve using:
Effective lateral stress \((\sigma_b') = 400\) kPa, i.e. \(k_o = 0.9\)
Friction angle \(\phi=\phi_{ev}\) of 34 degrees
Figure 8  Proposed stress ratio / shear strain curve

Figure 9  Volume strain / shear strain for proposed stress ratio curve in Figure 8
Figure 10  Predicted pressure/expansion curve based on the model given in Figure 8
APPENDIX F

Phase I U.B.C. Laboratory Testing Results
(Vaid et al., 1996)

N.B. Some errors were found in a few figures in the Vaid et al. (1996) report. These errors have been corrected and the revised figures (marked *CORRECTED*) are included in this report.
NOTES:

- Void ratio computations using $G_s = 2.62$. This was the value provided initially before a new value was specified. It was also the value measured at UBC.

- There might be a possible slip between sand and the loading platten at larger strains in test SYNO1.

- AP - Air pluviated
  WP - Water pluviated
  MT - moist tamped
  C - Contractive (strain softening)
  D - Dilative (strain hardening)
  TC - Triaxial compression
  TE - Triaxial extension

- Sand was boiled prior to reconstitution of specimens.
a) Reconstituted samples of isotropically consolidated and anisotropically consolidated Phase I sand tested in triaxial compression and extension
### CANLEX - PHASE I

**Monotonic Triaxial Tests on Reconstituted Syncrude Sand**

<table>
<thead>
<tr>
<th>Specimen No</th>
<th>$e_i$</th>
<th>$e_{20\text{kPa}}$</th>
<th>$\sigma_{ve}^c$ (kPa)</th>
<th>$\sigma_{ve}^{sc}$ (kPa)</th>
<th>$e_c$</th>
<th>Test Type</th>
<th>Contraction Dilatant</th>
<th>$S_{PT/DSS}$ (kPa)</th>
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<tbody>
<tr>
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<td>12</td>
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<td>C</td>
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</table>
### CANLEX - Phase I

Monotonic Triaxial Tests on Reconstituted Syncrude Sand (Contd..)

<table>
<thead>
<tr>
<th>Specimen No</th>
<th>$e_i$</th>
<th>$e_{20kPa}$</th>
<th>$\sigma'_{ve}$ (kPa)</th>
<th>$\sigma'_{hr}$ (kPa)</th>
<th>$e_2$</th>
<th>Test Type</th>
<th>Contractive Dilative</th>
<th>$S_{YRES}$ (kPa)</th>
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<td>16</td>
<td>0.871</td>
<td>0.843</td>
<td>800</td>
<td>400</td>
<td>0.810</td>
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<td>79.0</td>
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</table>
COMPRESSION TEST ($\sigma'_3 = 50$ kPa & $K_c = 1.0$)

Specimen No. 1
EXTENSION TEST ($\sigma'_3 = 50$ kPa & $K_c = 1.0$) 

Specimen No. 2

![Graphs showing deviatoric stress, pore pressure, and $(\sigma'_1 - \sigma'_3)/2$ vs. axial strain.](image)
COMPRESSION TEST ($\sigma'_3 = 100 \text{ kPa} \& K_c = 1.0$) Specimen No. 3

[Graphs showing deviatoric stress vs. axial strain, pore pressure vs. axial strain, and $\left(\frac{\sigma'_1 - \sigma'_3}{2}\right)$ vs. $\left(\frac{\sigma'_1 + \sigma'_3}{2}\right)$]
EXTENSION TEST ($\sigma'_3 = 100$ kPa & $K_c = 1.0$)

Specimen No. 4

Graphs showing deviatoric stress, pure pressure, and $(\sigma'_1 - \sigma'_3)/2$ versus axial strain.
COMPRESSION TEST $\left( \sigma'_3 = 200 \text{kPa} \& K_c = 1.0 \right)$

Specimen No. 5
EXTENSION TEST ($\sigma'_3 = 400$ kPa & $K_c = 1.0$)

Specimen No. 8
EXTENSION TEST ($\sigma'_3 = 400$ kPa & $K_c = 1.5$)

Specimen No. 9
COMPRESSION TEST ($\sigma'_3 = 50$ kPa & $K_c = 2.0$)

Specimen No. 10

*CORRECTED*
EXTENSION TEST ($\sigma'_3 = 50$ kPa & $K_c = 2.0$)

Specimen No. 11
*CORRECTED*
COMPRESSION TEST ($\sigma'_3 = 100$ kPa & $Kc = 2.0$)

Specimen No. 12

- Deviatoric stress (kPa)
- Pore pressure (kPa)
- ($\sigma'_1 - \sigma'_3$)/2 vs ($\sigma'_1 + \sigma'_3$)/2 (kPa)
EXTENSION TEST ($\sigma'_3 = 100$ kPa & $K_c = 2.0$)

Specimen No. 13
COMPRESSION TEST ($\sigma'_{3} = 200$ kPa & $K_c = 2.0$)

Specimen No. 14

- Deviatoric stress (kPa)
- Pore pressure (kPa)
- $(\sigma'_{1} - \sigma'_{3})/2$ (kPa) vs $(\sigma'_{1} + \sigma'_{3})/2$ (kPa)
EXTENSION TEST ($\sigma_3' = 200$ kPa & $K_c = 2.0$)

Specimen No. 15
CORRECTED*
COMPRESSION TEST ($\sigma'_3 = 400$ kPa & $K_c = 2.0$)

Specimen No. 16

- Deviatoric stress (kPa)

- Pore pressure (kPa)

- $(\sigma'_1 - \sigma'_3)/2$ (kPa)

- $(\sigma'_1 + \sigma'_3)/2$ (kPa)
EXTENSION TEST ($\sigma_3' = 400$ kPa & $K_c = 2.0$)

Specimen No. 17
Triaxial Test
Reconstituted Syncrude Sand

Monotonic Loading

Test No: S02

\( \sigma'_{vc} = 200 \text{ kPa} \)
\( \sigma'_{hc} = 200 \text{ kPa} \)
\( e_c = 0.831 \)
Monotonic Loading

Test No: S04

Reconstituted Syncrude Sand

\[ \sigma_{vc} = 200 \text{ kPa} \]

\[ \sigma_{hc} = 200 \text{ kPa} \]

\[ e_c = 0.843 \]
Triaxial Test
Reconstituted Syncrude Sand

Monotonic Loading
Test No: CC1

\[ \sigma_{vc} = 400 \text{ kPa} \]
\[ \sigma_{hc} = 400 \text{ kPa} \]
\[ e_c = 0.825 \]
Triaxial Test
Reconstituted Syncrude Sand

Monotonic Loading
Test No: CE1

\( \sigma'_{vc} = 400 \text{ kPa} \)
\( \sigma'_{hc} = 400 \text{ kPa} \)
\( e_c = 0.830 \)

\( \sigma_d \cdot \text{kPa} \)

\( \Delta u / \sigma'_{hc} \)

\( \varepsilon_a \)

\( (\sigma'_1 - \sigma'_3)/2 \cdot \text{kPa} \)

\( (\sigma'_1 + \sigma'_3)/2 \cdot \text{kPa} \)
Triaxial Test
Reconstituted Syncrude Sand

Monotonic Loading
Test No: CC5

\( \sigma'_{vc} = 400 \text{ kPa} \)
\( \sigma'_{hc} = 400 \text{ kPa} \)
\( e_c = 0.818 \)
Triaxial Test
Reconstituted Syncrude Sand

Monotonic Loading

Test No: CE3

\( \sigma'_v = 400 \text{ kPa} \)
\( \sigma'_h = 400 \text{ kPa} \)
\( e_c = 0.819 \)

Necking

\( \sigma_d, \text{kPa} \)

\( \Delta u/\sigma'_h \)

\( \varepsilon_a \)

\( (\sigma'_1 - \sigma'_3)/2, \text{kPa} \)

\( (\sigma'_1 + \sigma'_3)/2, \text{kPa} \)
Monotonic Triaxial Test on Frozen Moist Tamped Syncrude Sand

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sample No</th>
<th>e_i</th>
<th>Thaw</th>
<th>B (%)</th>
<th>e_20 kPa</th>
<th>σ' vc (kPa)</th>
<th>σ' hs (kPa)</th>
<th>Consolidation</th>
<th>e_c</th>
<th>Test Type</th>
<th>Contractive Dilatation</th>
<th>S_Press (kPa)</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Δh (mm)</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Undrained Tests</td>
<td>Frozen Moist Tamped</td>
<td>MT1</td>
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<td>0.990</td>
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<td>200</td>
<td>0.170</td>
<td>2.850</td>
<td>0.688</td>
<td>TC</td>
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</table>

Notes:
- Axial Strain during thaw = 5.04 %
- Void ratios are calculated using the dry weight of sand and dimensions
Triaxial Test
Frozen Moist Tamped

Monotonic Loading
Test No: MT1

Sample Collapsed during Thaw.

\( \sigma'_{vc} = 200 \text{ kPa} \)
\( \sigma'_{hc} = 200 \text{ kPa} \)
\( e'_{c} = 0.688 \)
**CANLEX - PHASE I**

**Monotonic Triaxial Tests on Frozen Water Pluviated Syncrude Sand**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sample No</th>
<th>e&lt;sub&gt;i&lt;/sub&gt;</th>
<th>Thaw</th>
<th>R (%)</th>
<th>e&lt;sub&gt;20&lt;/sub&gt; (kPa)</th>
<th>σ&lt;sub&gt;vc&lt;/sub&gt; (kPa)</th>
<th>σ&lt;sub&gt;hc&lt;/sub&gt; (kPa)</th>
<th>Consolidation</th>
<th>e&lt;sub&gt;c&lt;/sub&gt;</th>
<th>Test Type</th>
<th>Constrictive</th>
<th>S&lt;sub&gt;PTSS&lt;/sub&gt; (kPa)</th>
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<tr>
<td></td>
<td></td>
<td></td>
<td>Δh (mm)</td>
<td></td>
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<td></td>
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<tr>
<td>Undrained Tests</td>
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<td></td>
<td></td>
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<tr>
<td>Recons</td>
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<td>100</td>
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<td>400</td>
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<td>400</td>
<td>0.233</td>
<td>0.697</td>
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File: C:\CANLEX\FINAL\T-RF-P1.WP6

Creation Date: Apr 1996
Sample: 6-1

$e_z = 0.699$

$\sigma_{vc} = 400 \text{ kPa}$

$\sigma_{hc} = 400 \text{ kPa}$

$\left(\sigma_v - \sigma_h\right)_t$

$\Delta l / \sigma_{hc}$

$\varepsilon_{\sigma_i}, \%$

$\left(\sigma_v + \sigma_h\right)_t / 2$, kPa

$\left(\sigma_v - \sigma_h\right)_t$, kPa
Sample: 6-2

\( \varepsilon_c = 0.697 \)
\( \sigma_{ve} = 400 \text{ kPa} \)
\( \sigma_{hc} = 400 \text{ kPa} \)
b) Reconstituted samples of Phase I sand tested in hollow cylinder torsion
### Hollow Cylinder Torsion Tests on Reconstituted Syncrude Sand

<table>
<thead>
<tr>
<th>Test number</th>
<th>e after preparation</th>
<th>e after vacuum of 2 kPa</th>
<th>e after consolidation at 20 kPa</th>
<th>Stress path at 400 kPa</th>
<th>$\Phi_{CSR}$ (deg)</th>
<th>$\Phi_{PT}$ (deg)</th>
<th>$\Phi_f$ (deg)</th>
<th>Liq.</th>
<th>$S_u$(ss) (kPa)</th>
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<tbody>
<tr>
<td>Syn08</td>
<td>0.850</td>
<td>0.814</td>
<td>0.783</td>
<td>0.5, 45</td>
<td>19.2</td>
<td>35.4</td>
<td>-</td>
<td>L</td>
<td>39.0</td>
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<tr>
<td>Syn09</td>
<td>0.848</td>
<td>0.818</td>
<td>0.790</td>
<td>0.5, 30</td>
<td>22.1</td>
<td>34.6</td>
<td>-35.0</td>
<td>L.L</td>
<td>-</td>
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<td>Syn10</td>
<td>0.851</td>
<td>0.819</td>
<td>0.787</td>
<td>0.5, 60</td>
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<td>45.0</td>
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<td>&gt;45</td>
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<td>35.2</td>
<td>D</td>
<td>-</td>
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<td>30.7</td>
<td>32.8</td>
<td>L.L</td>
<td>-</td>
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</table>

All test specimen were isotropically consolidated with an effective stress of 400 kPa against a back pressure of 100 kPa. Tests Syn08, Syn09, Syn10 and Syn15 were performed by controlling shear strain at a rate of 0.1 %/min. Tests Syn13 and Syn14 were performed by controlling axial strain at a rate of 0.1 %/min.

### Notation:
- **L** - liquefaction
- **LL** - limited liquefaction
- **$\Phi_{CSR}$** - friction angle at peak
- **$\Phi_{PT}$** - friction angle at Phase transformation
- **$b$** - \( (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3) \)
- **$\alpha_o$** - inclination of $\sigma_1$ to vertical deposition direction
- **$\sigma_m$** - mean normal stress
- **$\sigma_{mc}'$** - mean normal consolidation
- **$\varepsilon_1$** - major principal strain
- **$D_r$** - relative density based on: $e_{max} = 0.968$, $e_{min} = 0.522$
$\sigma_{mc} = 400 \text{ kPa}, \ b = 0.5$

$\frac{(\sigma'_1 - \sigma'_3)}{2} (\text{kPa})$

$\frac{(\sigma'_1 + \sigma'_2 + \sigma'_3)}{3} (\text{kPa})$

$\sigma_1 - \sigma_3 (\text{kPa})$

Excess P.P. (kPa)

$\varepsilon_1 (\%)$

$\alpha_0 = 0, \ e_c = 0.793$

$30, 0.790$

$45, 0.783$

$60, 0.787$

$90, 0.796$
\( \sigma'_{me} = 400 \text{ kPa}, \ b = 0.5, \ D_r = 40 \% \)

\( \sigma_m, \ b \) and \( \alpha_\sigma \) held constant during shear

(a)

(b)

90
c) Reconstituted samples of Phase I sand tested in simple shear and cyclic simple shear
## Simple Shear Tests on Reconstituted Syncrude Sand

<table>
<thead>
<tr>
<th>Specimen No</th>
<th>Pluviation Method</th>
<th>$e_i$</th>
<th>$e_{20}$ (kPa)</th>
<th>$\sigma'_{v,c}$ (kPa)</th>
<th>$\tau_{static}$ (kPa)</th>
<th>$e_c$</th>
<th>Contractive Dilative</th>
<th>$S_{PTSS}$ (kPa)</th>
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<td>AP</td>
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<td>AP</td>
<td>0.846</td>
<td>200</td>
<td>0</td>
<td>0.778</td>
<td>C</td>
<td>12.1</td>
<td></td>
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<tr>
<td>SYN01</td>
<td>AP</td>
<td>0.051</td>
<td>400</td>
<td>0</td>
<td>0.773</td>
<td>C</td>
<td>29.0</td>
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</tr>
<tr>
<td>SYN02</td>
<td>AP</td>
<td>0.942</td>
<td>200</td>
<td>0</td>
<td>0.814</td>
<td>C</td>
<td>8.5</td>
<td></td>
</tr>
<tr>
<td>SYN03</td>
<td>AP</td>
<td>0.928</td>
<td>100</td>
<td>0</td>
<td>0.822</td>
<td>C</td>
<td>7.0</td>
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</tr>
<tr>
<td>SYN04</td>
<td>AP</td>
<td>0.934</td>
<td>50</td>
<td>0</td>
<td>0.842</td>
<td>C</td>
<td>4.2</td>
<td></td>
</tr>
<tr>
<td>WP01</td>
<td>WP</td>
<td>0.734</td>
<td>200</td>
<td>0</td>
<td>0.692</td>
<td>D</td>
<td></td>
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<tr>
<td>WP02</td>
<td>WP</td>
<td>0.764</td>
<td>200</td>
<td>0</td>
<td>0.744</td>
<td>D</td>
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<td>WP03</td>
<td>WP</td>
<td>0.799</td>
<td>200</td>
<td>0</td>
<td>0.778</td>
<td>D</td>
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<td></td>
</tr>
<tr>
<td>WP04</td>
<td>WP</td>
<td>0.760</td>
<td>200</td>
<td>0</td>
<td>0.721</td>
<td>D</td>
<td></td>
<td></td>
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<tr>
<td>WP05</td>
<td>WP</td>
<td>0.714</td>
<td>200</td>
<td>0</td>
<td>0.667</td>
<td>D</td>
<td></td>
<td></td>
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<tr>
<td>CT01</td>
<td>WP</td>
<td>0.916</td>
<td>400</td>
<td>0</td>
<td>0.742</td>
<td>D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CT02</td>
<td>WP</td>
<td>0.901</td>
<td>400</td>
<td>0</td>
<td>0.734</td>
<td>D</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

C:\CANLEX\SUMMARY\S-R-PJ.WP6 with Gs = 2.62
Simple Shear Test

Test No: AP01

\[ e_c = 0.809 \]
\[ \sigma_{ve} = 200 \text{ kPa} \]
Simple Shear Test

Test No: AP02

\[ \varepsilon = 0.751 \]
\[ \sigma_{ve} = 200 \text{ kPa} \]
Simple Shear Test

Test No: AP03

\[ e' = 0.791 \]
\[ \sigma_{vc} = 200 \text{ kPa} \]

\[ T_\tau, \text{ kPa} \]

\[ \Delta \mu / \sigma_{vc} \]

\[ T_\tau, \text{ kPa} \]

\[ \gamma, \% \]
Simple Shear Test

Test No: AP04

e_c = 0.745
\sigma_{vc} = 200 \text{ kPa}
Simple Shear Test

Test No: AP05

\[ \varepsilon_s = 0.653 \]
\[ \sigma_{vo} = 200 \text{ kPa} \]
Simple Shear Test

Test No: AP06

\( \varepsilon_e = 0.721 \)

\( \sigma_v = 200 \text{ kPa} \)

---

Graphs showing the relationship between stress and strain for the Simple Shear Test.
Simple Shear Test

Test No: AP07

e_c = 0.735
σ_v = 200 kPa

\( \tau, \text{kPa} \)

\( \Delta \ln/\sigma_v \)

\( \sigma_v, \text{kPa} \)
Simple Shear Test

Test No: AP08

\[ e_c = 0.763 \]
\[ \sigma_{vc} = 200 \text{ kPa} \]

![Graph of shear stress \( \tau \) versus shear strain \( \gamma \)]

![Graph of relative density change \( \Delta \rho / \rho_c \) versus shear strain \( \gamma \)]

![Graph of shear stress \( \tau \) versus effective vertical stress \( \sigma'_{vc} \)]
Simple Shear Test

Test No: AP09

\[ e_c = 0.777 \]

\[ \sigma_{vc} = 200 \text{ kPa} \]
Simple Shear Test

Test No : AP10

\[ e_s = 0.778 \]
\[ \sigma_{ve} = 200 \text{ kPa} \]
Simple Shear Test

Test No: SYN01

\[ \varepsilon_c = 0.773 \]
\[ \sigma_{ve} = 400 \text{ kPa} \]
Simple Shear Test

Test No: SYN02

\( e_c = 0.814 \)

\( \sigma_{ve} = 200 \text{ kPa} \)
Simple Shear Test

Test No: SYN03

e_c = 0.822
\sigma_{vc} = 100 \text{ kPa}
Simple Shear Test

Test No: SYN04

\[ e_c = 0.842 \]
\[ \sigma_{ve} = 50 \text{ kPa} \]

\begin{align*}
\tau, \text{ kPa} & \\
\Delta u/\sigma_{ve} & \\
\sigma_{\nu}, \text{ kPa} & 
\end{align*}

\begin{align*}
0 & \quad 2 \quad 4 \quad 6 \quad 8 \quad 10 \\
0 & \quad 0.2 \quad 0.4 \quad 0.6 \quad 0.8 \quad 1.0 \\
0 & \quad 5 & \quad 10 & \quad 15 & \quad 20 & \quad 25
\end{align*}
Simple Shear Test

Test No: WP01

e_c = 0.692
σ'_c = 200 kPa

\[ \tau, \text{kPa} \]

\[ \Delta u/\sigma'_c \]

\[ \tau, \text{kPa} \]

\[ \sigma'_v, \text{kPa} \]
Simple Shear Test

Test No: WP02

$e_c = 0.744$

$\sigma'_c = 200 \text{ kPa}$

---

Graph 1: $\tau$, kPa vs $\gamma$, %

Graph 2: $\Delta H / \sigma_{wc}$ vs $\gamma$, %

Graph 3: $\tau$, kPa vs $\sigma'_v$, kPa
Simple Shear Test

Test No: WP03

\[ e_c = 0.778 \]
\[ \sigma_c = 200 \text{ kPa} \]

\( \tau, \text{kPa} \)
\( \Delta u/\sigma_c \)

\( \gamma, \text{}\)
Simple Shear Test

Test No: WP04

\[ \epsilon = 0.721 \]

\[ \sigma_c' = 200 \text{ kPa} \]
Simple Shear Test

Test No: WP05

\[ e_{c*} = 0.667 \]
\[ \sigma_{c} = 200 \text{ kPa} \]
Simple Shear Test

Test No: CT01

\[ \sigma_c' = 400 \text{ kPa} \]

\[ e_c = 0.742 \]
Simple Shear Test  

Test No: CT02

\[ \sigma' = 400 \text{ kPa} \]
\[ e_\varepsilon = 0.734 \]

\[ \gamma, \% \]

\[ \Delta \varepsilon / \sigma' \varepsilon \]

\[ \tau, \text{ kPa} \]

\[ \sigma'_v, \text{ kPa} \]
## CANLEX - PHASE I

### Monotonic Simple Shear Tests on Reconstituted Syncrude Sand

(Continued)

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test ID</th>
<th>$e_i$</th>
<th>$e_{20\text{P}}$</th>
<th>$\sigma'_v$ (kPa)</th>
<th>$\tau_d/\sigma'_v$</th>
<th>$e_c$</th>
<th>Static Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S03</td>
<td>0.917</td>
<td>0.811</td>
<td>50</td>
<td>0.0</td>
<td>0.788</td>
<td>C</td>
</tr>
<tr>
<td>2</td>
<td>S06</td>
<td>0.918</td>
<td>0.801</td>
<td>50</td>
<td>-0.3</td>
<td>0.774</td>
<td>D</td>
</tr>
<tr>
<td>3</td>
<td>S07</td>
<td>0.926</td>
<td>0.792</td>
<td>100</td>
<td>0.0</td>
<td>0.774</td>
<td>C</td>
</tr>
<tr>
<td>4</td>
<td>S08</td>
<td>0.905</td>
<td>0.796</td>
<td>100</td>
<td>0.3</td>
<td>0.762</td>
<td>C</td>
</tr>
<tr>
<td>5</td>
<td>S12</td>
<td>0.882</td>
<td>0.809</td>
<td>100</td>
<td>-0.3</td>
<td>0.772</td>
<td>D</td>
</tr>
<tr>
<td>6</td>
<td>S09</td>
<td>0.928</td>
<td>0.793</td>
<td>200</td>
<td>0.0</td>
<td>0.763</td>
<td>C</td>
</tr>
<tr>
<td>7</td>
<td>S15</td>
<td>0.915</td>
<td>0.790</td>
<td>200</td>
<td>0.3</td>
<td>0.755</td>
<td>C</td>
</tr>
<tr>
<td>8</td>
<td>S16</td>
<td>0.889</td>
<td>0.797</td>
<td>200</td>
<td>-0.3</td>
<td>0.757</td>
<td>C</td>
</tr>
<tr>
<td>9</td>
<td>S17</td>
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<td>0.787</td>
<td>400</td>
<td>0.0</td>
<td>0.736</td>
<td>C</td>
</tr>
<tr>
<td>10</td>
<td>S19</td>
<td>0.907</td>
<td>0.807</td>
<td>400</td>
<td>0.3</td>
<td>0.746</td>
<td>D</td>
</tr>
<tr>
<td>11</td>
<td>S20</td>
<td>0.894</td>
<td>0.802</td>
<td>400</td>
<td>-0.3</td>
<td>0.740</td>
<td>D</td>
</tr>
</tbody>
</table>

$e_i$, $e_{20\text{P}}$, $e_c$ : Calculated from final dry weight of solids and dimensions

File: C:\CANLEX\RECONS\PHASE.I\MON-P1R.WP6

Created On: Apr 20, 1996.
**CANLEX - PHASE I**

Monotonic Simple Shear Tests on Reconstituted Syncrude Sand

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test ID</th>
<th>$e_i$</th>
<th>$e_{20 kPa}$</th>
<th>$\sigma_{ve}'$ (kPa)</th>
<th>$\tau_d/\sigma_{ve}'$</th>
<th>$e_o$</th>
<th>Static Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>S01</td>
<td>0.868</td>
<td>0.768</td>
<td>50</td>
<td>0.0</td>
<td>0.752</td>
<td>D</td>
</tr>
<tr>
<td>14</td>
<td>S04</td>
<td>0.862</td>
<td>0.761</td>
<td>50</td>
<td>0.0</td>
<td>0.748</td>
<td>D</td>
</tr>
<tr>
<td>15</td>
<td>S05</td>
<td>0.879</td>
<td>0.771</td>
<td>50</td>
<td>0.0</td>
<td>0.759</td>
<td>D</td>
</tr>
<tr>
<td>16</td>
<td>S02</td>
<td>0.877</td>
<td>0.746</td>
<td>100</td>
<td>0.0</td>
<td>0.723</td>
<td>D</td>
</tr>
<tr>
<td>17</td>
<td>S10</td>
<td>0.864</td>
<td>0.774</td>
<td>100</td>
<td>0.3</td>
<td>0.735</td>
<td>D</td>
</tr>
</tbody>
</table>

$e_i$, $e_{20kPa}$, $e_o$: Calculated from final dry weight of solids and dimensions.

File: C:\CANLEX\RECONS\PHASE.I\MON-P1R.WP6

Created On: Apr 20, 1996.
Simple Shear Test

Reconstituted Syncrude Sand

\( \sigma'_{vc} = 50 \text{ kPa} \)
\( e_c = 0.788 \)
\( \tau_{st} / \sigma'_{vc} = 0 \)

Monotonic Loading

Test No: S03

\( \Delta u / \sigma'_{vc} \)

\( \tau, \text{kPa} \)

\( \sigma'_{v}, \text{kPa} \)
Simple Shear Test

Reconstituted Syncrude Sand

\[ \sigma'_{vc} = 50 \text{ kPa} \]
\[ e_c = 0.771 \]
\[ \tau_{st} / \sigma'_{vc} = 0.30 \]

Monotonic Loading

Test No: S06
Simple Shear Test

Reconstituted Syncrude Sand

Test No: S07

\[ \sigma'_{vc} = 50 \text{ kPa} \]

\[ e_c = 0.774 \]

\[ \tau_{st} / \sigma'_{vc} = -0.30 \]
Simple Shear Test

Reconstituted Syncrude Sand

Test No: S08

\[ \sigma_{vc} = 100 \text{ kPa} \]
\[ e_c = 0.774 \]
\[ \tau_{sl} / \sigma_{vc} = 0 \]

\( \tau, \text{kPa} \)

\( \Delta u / \sigma_{vo} \)

\( \gamma, \% \)

\( \sigma_{vc}, \text{kPa} \)

\( \tau, \text{kPa} \)
Simple Shear Test

Monotonic Loading

Reconstituted Syncrude Sand

Test No: S12

\( \sigma'_{vc} = 100 \text{ kPa} \)

\( e_c = 0.762 \)

\( \tau_{st} / \sigma'_{vc} = 0.30 \)

\( \tau, \text{ kPa} \)

\( \Delta u / \sigma'_{vo} \)

\( \gamma, \% \)

\( \tau, \text{ kPa} \)

\( \sigma'_{vo}, \text{ kPa} \)
Simple Shear Test

Monotonic Loading

Reconstituted Syncrude Sand

Test No: S09

\( \sigma'_{vc} = 100 \text{ kPa} \)

\( e_c = 0.772 \)

\( \tau_{st} / \sigma'_{vc} = -0.30 \)
Simple Shear Test

Reconstituted Syncrude Sand Test No: S15

$\sigma'_{vc} = 200$ kPa
$e_c = 0.763$
$\tau_{st}/\sigma'_{vc} = 0$

Monotonic Loading

$\tau$, kPa

$\Delta u/\sigma'_{vc}$

$\gamma$, %

$\tau$, kPa

$\sigma'_v$, kPa

122
Simple Shear Test

Reconstituted Syncrude Sand

\( \sigma_{vc} = 200 \text{ kPa} \)
\( e_c = 0.755 \)
\( \tau_{st} / \sigma_{vc} = 0.30 \)

Monotonic Loading

Test No: S17
Simple Shear Test

Monotonic Loading

Reconstituted Syncrude Sand

Test No: S16

\[ \sigma'_{vc} = 200 \text{ kPa} \]
\[ e_c = 0.757 \]
\[ \frac{\tau_{sl}}{\sigma'_{vc}} = -0.30 \]

\[ \tau, \text{ kPa} \]

\[ \Delta \psi/\sigma'_{vo} \]

\[ \gamma, \% \]

\[ \tau, \text{ kPa} \]

\[ \sigma'_{v}, \text{ kPa} \]
Simple Shear Test

Reconstituted Syncrude Sand

Test No: S19

\( \sigma'_{vc} = 400 \text{ kPa} \)
\( e_c = 0.736 \)
\( \tau_{st}/\sigma'_{vc} = 0 \)

\( \Delta\mu/\sigma'_{vo} \)

\( \tau, \text{ kPa} \)

\( \sigma', \text{ kPa} \)
Simple Shear Test

Reconstituted Syncrude Sand

\[ \sigma'_v = 400 \, \text{kPa} \]

\[ e_c = 0.746 \]

\[ \tau_{st} / \sigma'_v = 0.30 \]

Monotonic Loading

Test No: S20
Simple Shear Test

Reconstituted Syncrude Sand

Test No: S18

- $\sigma'_v = 400$ kPa
- $e_c = 0.740$
- $\tau_{sf} / \sigma'_v = -0.30$

Graphs showing:
- Stress-strain relationship
- Strain vs. strain
- Stress vs. effective stress
Simple Shear Test

Monotonic Loading

Reconstituted Syncrude Sand
Test No: S01

$\sigma'_{vc} = 50 \text{ kPa}$
$e_c = 0.752$
$\tau_{sl}/\sigma'_{vc} = 0$

Graphs showing:
- $\tau$, kPa vs $\gamma$, %
- $\Delta u/\sigma'_{vc}$ vs $\gamma$, %
- $\tau$, kPa vs $\sigma'_{vc}$, kPa
Simple Shear Test

Reconstituted Syncrude Sand

Test No : S04

\[ \sigma'_{vc} = 50 \text{kPa} \]

\[ e_c = 0.748 \]

\[ \tau_{st}/\sigma'_{vc} = 0 \]

\[ \Delta u/\sigma'_{vo} \]

\[ \gamma, \% \]

\[ \tau', \text{kPa} \]

\[ \sigma'_{v'}, \text{kPa} \]
Simple Shear Test  

Monotonic Loading

Reconstituted Syncrude Sand

Test No: S05

\[ \sigma'_{ve} = 50 \text{ kPa} \]
\[ e_c = 0.759 \]
\[ \tau_{nl} / \sigma'_{ve} = 0 \]

\[ \tau, \text{kPa} \]

\[ \Delta u / \sigma'_{ve} \]

\[ \gamma, \% \]

\[ \sigma'_{ve}, \text{kPa} \]

\[ \tau, \text{kPa} \]

130
Simple Shear Test

Monotonic Loading

Reconstituted Syncrude Sand

Test No: S02

$\sigma_{vc} = 100 \text{ kPa}$

$e_c = 0.723$

$\tau_{stl} / \sigma_{vc} = 0$

$\tau, \text{ kPa}$

$\Delta u / \sigma_{vo}$

$\sigma'_{v}, \text{ kPa}$

$\gamma, \%$
Simple Shear Test

Monotonic Loading

Reconstituted Syncrude Sand

Test No: S10

\( \sigma'_{sv} = 100 \text{ kPa} \)

\( e_c = 0.735 \)

\( \tau_{st} / \sigma'_{sv} = 0.30 \)
Canlex - Phase I

Cyclic Simple Shear Tests on Reconstituted syncrude Sand

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test ID</th>
<th>$e_i$</th>
<th>$e_{20$kPa}$</th>
<th>$\sigma_{ve}'$ (kPa)</th>
<th>$\tau_{static}$ (kPa)</th>
<th>$e_e$</th>
<th>$\tau_{ey}/\sigma_{ve}'$</th>
<th>N</th>
<th>$\gamma_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S11</td>
<td>0.884</td>
<td>0.743</td>
<td>400</td>
<td>0</td>
<td>0.703</td>
<td>0.117</td>
<td>35</td>
<td>6</td>
</tr>
<tr>
<td>2</td>
<td>S13</td>
<td>0.868</td>
<td>0.767</td>
<td>400</td>
<td>0</td>
<td>0.728</td>
<td>0.160</td>
<td>2</td>
<td>7.8</td>
</tr>
<tr>
<td>3</td>
<td>S14</td>
<td>0.859</td>
<td>0.759</td>
<td>400</td>
<td>0</td>
<td>0.725</td>
<td>0.141</td>
<td>4</td>
<td>7.6</td>
</tr>
</tbody>
</table>
Simple Shear Test  Cyclic Loading

Test No: S11

Reconstituted Syncrude Sand

\[ \sigma'_{vc} = 400 \text{kPa} \]

\[ e_c = 0.703 \]

\[ \tau_{cy} / \sigma'_{vc} = 0.117 \]

\[ \Delta u / \sigma'_{vc} \]

\[ \gamma, \% \]

No of Cycles
Simple Shear Test

Cyclic Loading

Reconstituted Syncrude Sand

Test No: S13

\[ \sigma'_{vc} = 400 \text{ kPa} \]

\[ e_c = 0.728 \]

\[ \tau_{cy}/\sigma'_{vc} = 0.160 \]

No of Cycles

1.0

0.5

0.0

8

4

0

-4

-8

0 1 2 3 4 5

No of Cycles
Simple Shear Test

Reconstituted Syncrude Sand

Test No: S14

\[ \sigma'_{vc} = 400 \text{ kPa} \]

\[ e_c = 0.725 \]

\[ \tau_{cy} / \sigma'_{vc} = 0.141 \]

\[ \Delta u / \sigma'_{vc} \]

\[ \gamma \%

No of Cycles

136
## Monotonic Simple Shear Tests on Frozen Moist Tamped Syncrude Sand

<table>
<thead>
<tr>
<th>Specimen No</th>
<th>Method of Reconstitution</th>
<th>$e_i$</th>
<th>$e_{20\text{kPa}}$</th>
<th>$\sigma_{ve}^\prime$ (kPa)</th>
<th>$\tau_{\text{static}}$ (kPa)</th>
<th>$e_c$</th>
<th>Contractive Dilative</th>
<th>$S_{PT/SS}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DS#1</td>
<td>MT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DS#2</td>
<td>MT</td>
<td>1.056</td>
<td>0.937</td>
<td>200</td>
<td>0</td>
<td>0.768</td>
<td>C</td>
<td>3.7</td>
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<tr>
<td>DS#3</td>
<td>MT</td>
<td>1.048</td>
<td>0.943</td>
<td>200</td>
<td>0</td>
<td>0.764</td>
<td>C</td>
<td>3.8</td>
</tr>
</tbody>
</table>

Notes:
- Specimen DS#1 collapsed during thaw with very large axial strain.
- Void ratios are calculated using the dry weight of sand and dimensions.
- Large strains during thaw and consolidation.
Simple Shear Test

Test No: DS#2

\[ \sigma'_c = 200 \text{ kPa} \]
\[ e_c = 0.768 \]

\( \tau, \text{kPa} \)

\( \Delta t / \sigma_w' \)

\( \gamma, \% \)

\( \tau, \text{kPa} \)

\( \sigma'_v, \text{kPa} \)
Simple Shear Test

Test No: DS#3

\[ \sigma_c' = 200 \text{ kPa} \]
\[ e_c = 0.764 \]

\( T_1, \text{kPa} \)

\( \Delta h / \sigma'_{vc} \)

\( \sigma'_v, \text{kPa} \)
## Monotonic Simple Shear Tests on Frozen Water Pluviation Syenite Sand

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sample No</th>
<th>$e_i$</th>
<th>$e_{20\text{kPa}}$</th>
<th>$\sigma'_{ve}$ (kPa)</th>
<th>$\tau_{\text{static}}$ (kPa)</th>
<th>$e_c$</th>
<th>Static Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reconstituted</td>
<td>6-3</td>
<td>0.746</td>
<td>0.752</td>
<td>400</td>
<td>0</td>
<td>0.716</td>
<td>D</td>
</tr>
<tr>
<td>Reconstituted</td>
<td>6-4</td>
<td>0.747</td>
<td>0.724</td>
<td>400</td>
<td>0</td>
<td>0.688</td>
<td>D</td>
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</table>
\[
\sigma_{ve} = 400 \text{ kPa} \\
e_c = 0.716
\]
SS (CV)  
Sample: 6-4

\[ \sigma_{vc}' = 400 \text{ kPa} \]
\[ e_c = 0.688 \]

\[ \Delta M / \sigma_{vc}' \]

\[ \gamma, \% \]

\[ T, \text{ kPa} \]

\[ \sigma_{vc}', \text{ kPa} \]

T02

142
d) Undisturbed Phase I samples tested in triaxial compression and extension, and simple shear
## CANLEX - PHASE I

### Monotonic Triaxial Tests on Frozen Syncrude Sand

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sample No</th>
<th>$e_i$</th>
<th>Thaw $\Delta h$ (mm)</th>
<th>$B$ (%)</th>
<th>$e$ 20 kPa</th>
<th>$\sigma_{vc}^1$ (kPa)</th>
<th>$\sigma_{hc}^1$ (kPa)</th>
<th>Consolidation $\Delta h$ (mm)</th>
<th>$e_c$</th>
<th>Test Type</th>
<th>Contractive Dilative</th>
<th>$S_{PTSS}$ (kPa)</th>
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</thead>
<tbody>
<tr>
<td><strong>Undrained Tests</strong></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>30.83</td>
<td>FS3 C7A</td>
<td>0.753</td>
<td>0.90</td>
<td>100</td>
<td>0.697</td>
<td>510</td>
<td>255</td>
<td>0.918</td>
<td>0.683</td>
<td>TC</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>36.30</td>
<td>FS5 C18 1</td>
<td>0.790</td>
<td>1.31</td>
<td>100</td>
<td>0.734</td>
<td>570</td>
<td>285</td>
<td>2.123</td>
<td>0.708</td>
<td>TC</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>36.44</td>
<td>FS5 C18 2</td>
<td>0.766</td>
<td>0.30</td>
<td>100</td>
<td>0.710</td>
<td>570</td>
<td>285</td>
<td>1.634</td>
<td>0.691</td>
<td>TE</td>
<td>C</td>
<td>38</td>
</tr>
<tr>
<td>35.10</td>
<td>FS4 C13A</td>
<td>0.802</td>
<td>0.97</td>
<td>100</td>
<td>0.736</td>
<td>550</td>
<td>275</td>
<td>1.042</td>
<td>0.718</td>
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<td>C</td>
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<td><strong>Drained Tests</strong></td>
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<td></td>
<td></td>
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<tr>
<td>31.80</td>
<td>FS1 C9B 1</td>
<td>0.759</td>
<td>1.06</td>
<td>100</td>
<td>0.702</td>
<td>540</td>
<td>270</td>
<td>1.285</td>
<td>0.684</td>
<td>TC</td>
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<tr>
<td>31.80</td>
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<td>0.773</td>
<td>1.00</td>
<td>100</td>
<td>0.717</td>
<td>540</td>
<td>270</td>
<td>2.491</td>
<td>0.694</td>
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<tr>
<td>33.84</td>
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<td>0.817</td>
<td>0.90</td>
<td>100</td>
<td>0.752</td>
<td>540</td>
<td>270</td>
<td>1.495</td>
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<td>0.75</td>
<td>100</td>
<td>0.693</td>
<td>540</td>
<td>270</td>
<td>1.596</td>
<td>0.675</td>
<td>TE</td>
<td>C</td>
<td>74</td>
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</table>
Sample: FS3 C7A

\[ e_c = 0.683 \]
\[ \sigma_{vc} = 510 \text{ kPa} \]
\[ \sigma_{hc} = 255 \text{ kPa} \]
$e_f = 0.708$

$\sigma_{vc} = 570 \text{ kPa}$

$\sigma_{he} = 285 \text{ kPa}$
$\theta_c = 0.691$

$\sigma_{rc} = 570 \text{ kPa}$

$\sigma_{nc} = 285 \text{ kPa}$
Sample: FS4 C13A

\( \sigma_v - \sigma_h \), kPa

\( \Delta \mu / \sigma_h \)

\( (\sigma_v + \sigma_h) / 2 \), kPa

\( e_s = 0.718 \)
\( \sigma'_v = 550 \) kPa
\( \sigma'_h = 275 \) kPa

Necking
Sample: FS1-C9B 1

\( e_c = 0.684 \)
\( \sigma_{vc} = 540 \text{ kPa} \)
\( \sigma_{hc} = 270 \text{ kPa} \)
TEA(D) Sample: FS1 C9B 2

\( \varepsilon_c = 0.694 \)
\( \sigma_{vc} = 540 \text{ kPa} \)
\( \sigma_{nc} = 270 \text{ kPa} \)

Necking

---

\( \sigma_1 / \sigma_3 \)

---

\( \varepsilon_a, \% \)

---

\( \varepsilon_c, \% \)
Sample: FS5 C13 1

\[ (\sigma_y - \sigma_n) \text{, kPa} \]

\[ e_c = 0.730 \]
\[ \sigma_{vc} = 540 \text{ kPa} \]
\[ \sigma_{hc} = 270 \text{ kPa} \]
Sample: FS5 C13 2

\( \varepsilon_c = 0.675 \)
\( \sigma_{ve} = 540 \text{ kPa} \)
\( \sigma_{hc} = 270 \text{ kPa} \)

Necking

\( \varepsilon_v, \% \)

\( \sigma_1/\sigma_3 \)
**CANLEX - PHASE I**

**Monotonic Simple Shear Tests on Frozen Syncrude Sand**

<table>
<thead>
<tr>
<th>Depth  (m)</th>
<th>Sample No</th>
<th>$e_i$</th>
<th>$e_{20, kPa}$</th>
<th>$\sigma_{ve}^t$ (kPa)</th>
<th>$\tau_{static}$ (kPa)</th>
<th>$e_c$</th>
<th>Static Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>NA</td>
<td>FS1 C3B 1</td>
<td>0.776</td>
<td>0.739</td>
<td>510</td>
<td>0</td>
<td>0.686</td>
<td>D</td>
</tr>
<tr>
<td>NA</td>
<td>FS1 C3B 2</td>
<td>0.754</td>
<td>0.725</td>
<td>510</td>
<td>0</td>
<td>0.682</td>
<td>D</td>
</tr>
<tr>
<td>31.141</td>
<td>FS3 C7B 1</td>
<td>0.858</td>
<td>0.817</td>
<td>520</td>
<td>0</td>
<td>0.777</td>
<td>C 57</td>
</tr>
<tr>
<td>31.181</td>
<td>FS3 C7B 2</td>
<td>0.791</td>
<td>0.782</td>
<td>520</td>
<td>0</td>
<td>0.737</td>
<td>C 88</td>
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<tr>
<td>33.201</td>
<td>FS1 C8B 1</td>
<td>0.793</td>
<td>0.772</td>
<td>540</td>
<td>0</td>
<td>0.727</td>
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<tr>
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<td>0.775</td>
<td>540</td>
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<td>0.724</td>
<td>D</td>
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<tr>
<td>29.631</td>
<td>FS4 C4A 1</td>
<td>0.773</td>
<td>0.763</td>
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<td>0</td>
<td>0.720</td>
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<td>29.661</td>
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<td>0.749</td>
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<td>510</td>
<td>0</td>
<td>0.709</td>
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</table>

Date: April 24, 1996
$\sigma_{ec} = 510 \text{ kPa}$

$e_c = 0.686$
SS (CV)  Sample: FS1-C3B (2)

\[ \sigma'_{ve} = 510 \text{ kPa} \]
\[ e_c = 0.682 \]
SS (CV) Sample: FS3-C7B (1)

\[ \sigma'_{rc} = 520 \text{ kPa} \]
\[ e_c = 0.777 \]
Sample: FS3-C7B (2)

\[ \sigma'_{ve} = 520 \text{ kPa} \]
\[ e_c = 0.737 \]
SS (CV) Sample: FS1-C8B (1)

\[ \sigma'_{ve} = 540 \text{ kPa} \]
\[ e_c = 0.727 \]
SS (CV) Sample: FS1-C8B (2)

\[ \sigma'_{vc} = 540 \text{ kPa} \]

\[ e_c = 0.724 \]
\[ \sigma_{ve}' = 510 \text{ kPa} \]
\[ e_c = 0.720 \]
APPENDIX G

Phase I Laval Laboratory Testing Results
(Konrad and Saint-Laurent, 1995)
<table>
<thead>
<tr>
<th>ESSAI</th>
<th>Nature</th>
<th>indice des vides</th>
<th>État critique</th>
<th>État</th>
<th>Variation charge verticale (kN)</th>
<th>Variation pression interstitiel (kPa)</th>
<th>$e(%)$</th>
<th>$q$</th>
<th>$p'$</th>
<th>$q/\sigma_c$</th>
<th>$p'/\sigma_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM01</td>
<td>Remanié</td>
<td>0,877</td>
<td>Contractant</td>
<td>Initial</td>
<td>0,0</td>
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<td>N/D</td>
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</table>

**TABLEAU B4** Résumé numérique de tous les essais triaxiaux
**Sable reconstitué de Fort McMurray**

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<thead>
<tr>
<th>Nature du sable</th>
<th>Sable de barrage sub-anguleux, uniformément gradué</th>
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<tbody>
<tr>
<td>Grosseur moyenne des grains</td>
<td>$D_{50}=0.15 \text{ mm}$</td>
</tr>
<tr>
<td>Pourcentage de particules fines</td>
<td>10 %</td>
</tr>
<tr>
<td>Densité spécifique</td>
<td>$G_s=2.63$</td>
</tr>
<tr>
<td>Coefficient d'uniformité</td>
<td>$C_U=2.3$</td>
</tr>
<tr>
<td>Diamètre effectif</td>
<td>$D_{10}=0.074$</td>
</tr>
<tr>
<td>Indice des vides min. et max.</td>
<td>$e_{\text{min}}=0.542$  $e_{\text{max}}=0.992$</td>
</tr>
</tbody>
</table>

**Sable intact (FS5 C5C) de Fort McMurray**

<table>
<thead>
<tr>
<th>Nature du sable</th>
<th>Sable de barrage sub-anguleux, uniformément gradué</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grosseur moyenne des grains</td>
<td>$D_{50}=0.17 \text{ mm}$</td>
</tr>
<tr>
<td>Pourcentage de particules fines</td>
<td>5 %</td>
</tr>
<tr>
<td>Densité spécifique</td>
<td>$G_s=2.63$</td>
</tr>
<tr>
<td>Coefficient d'uniformité</td>
<td>$C_U=1.9$</td>
</tr>
<tr>
<td>Diamètre effectif</td>
<td>$D_{10}=0.095$</td>
</tr>
<tr>
<td>Indice des vides min. et max.</td>
<td>$e_{\text{min}}=0.542$  $e_{\text{max}}=0.992$</td>
</tr>
</tbody>
</table>

**TABLEAU B1 Caractéristiques du sable de Fort McMurray**
Provenance: Fort McMurray
Échantillons intact et reconstitué
Profondeur: ------

Université Laval
Laboratoire de géotechnique
GRANULOMÉTRIE

Responsable: Jean-Marie Konrad
Technicien: Steve Saint-Laurent
Date: Avril 1995

% passant

État initial du sable
- Reconstitué
- Intact (Échantillon no. FSS-C5C)

Dimension des grains (millimètres)

FIG. B2 Granulométries du sable de Fort McMurray
a) Reconstituted (moist-tamped) samples of isotropically consolidated Phase I sand tested in triaxial compression
FIG. 3.3 Essais triaxiaux sur les échantillons reconstitués
FIG. 3.5 Enveloppe d'État critique du sable de Fort McMurray

\[
p'_{\text{normalisé}} = \frac{(\sigma_1 - \sigma_3)}{\sigma_c}
\]
Contrainte déviatorique

Cheminement des contraintes

Variation de la pression interstitielle

Essai triaxial Fort McMurray no.01
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 304 kPa  \( B : 1.000 \)

Caractéristiques de l'essai triaxial
Consolidation isotrope
Dim. de l'échant.[mm]: Diam.:100.0  Haut.:98.457
Indice des vides: \( e=0.877 \)
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
\( p'i=313.4 \text{ kPa} \quad p'f=0.0 \text{ Kpa} \)
\( qi=0 \text{ kPa} \quad qf=1.9 \text{ kPa} \)
Contrainte déviatorique

Cheminement des contraintes

Variation de la pression interstitielle

Essai triaxial Fort McMurray no.03
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 275 kPa  B : 0.986

Caractéristiques de l'essai triaxial
Consolidation isotrope
Dim. de l'échant.[mm]: Diam.:100.0  Haut.:99.344
Indice des vides: e=0.800
Type de chargement: Constant à 0.030 "/min.
Type de rupture: En tonneau
p'i=726.9 kPa  p'f=145.0 Kpa
qi=0 kPa  qf=229.6 kPa
Essai triaxial Fort McMurray no.04
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 279 kPa  $\bar{\varepsilon} : 0.991$

Caractéristiques de l'essai triaxial
Consolidation isotrope
Dim. de l'échantillon [mm]: Diam.:100.0  Haut.:99.425
Indice des vides: $e=0.819$
Type de chargement: Constant à 0.030 "/min.
Type de rupture: En tonneau
$p'i=294.6$ kPa  $p'f=18.1$ Kpa
$q'i=0$ kPa  $q'f=28.2$ kPa
Contrainte déviatorique

Cheminement des contraintes

Variation de la pression interstitielle

Essai triaxial Fort McMurray no.05
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 236 kPa  $B : 1.000$

Caractéristiques de l'essai triaxial
Consolidation isotrope
Dim. de l'échantillon [mm]: Diam.: 100.0 Haut.: 99.217
Indice des vides: $e = 0.810$
Type de chargement: Constant à 0.020 °/min.
Type de rupture: En tonneau
$p'i = 397.0 \text{kPa}$  $p'f = 19.7 \text{kPa}$
$q_i = 0 \text{kPa}$  $q_f = 30.5 \text{kPa}$
Contrainte déviatorique

Cheminement des contraintes

Variation de la pression interstielie

Essai triaxial Fort McMurray no.32
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 558 kPa   $B : 0.974$

Caractéristiques de l'essai triaxial
Consolidation isotope
Dim. de l'échant.[mm]: Diam.:100.0  Haut.:98.425
Indice des vides: $e=0.881$
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
$p'i=301.4$ kPa   $p'f=0.0$ Kpa
$q'i=0$ kPa   $q'f=0.5$ kPa
Essai triaxial Fort McMurray no. 33
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 502 kPa  B : 0.991

Caractéristiques de l'essai triaxial
Consolidation isotrope
Dim. de l'échantillon [mm]: Diam.: 100.0  Haut.: 98.314
Indice des vides: e = 0.797
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
p'i = 201.2 kPa  p'f = 52.2 KPa
qi = 0 kPa  qf = 83.7 kPa
Essai triaxial Fort McMurray no. 34
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 507 kPa \( B : 0.986 \)

Caractéristiques de l'essai triaxial
Consolidation isotope
Dim. de l'échant.[mm]: Diam.:100.0 Haut.:98.870
Indice des vides: e=0.864
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
\( p^i=201.6 \text{ kPa} \)
\( q_i=0 \text{ kPa} \)
\( p^f=0.6 \text{ kPa} \)
\( q_f=4.0 \text{ kPa} \)
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 596 kPa  B : 0.931

Caractéristiques de l'essai triaxial
Consolidation isotope
Dim. de l'échant.[mm]: Diam.:100.0  Haut.:99.287
Indice des vides: e=0.661
Type de chargement: Constant à 0.030 "/min.
Type de rupture: En tonneau
p'i=403.1 kPa  p'f=794.7 Kpa
qi=0 kPa  qf=1239.3 kPa
Essai triaxial Fort McMurray no. 36
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 621 kPa  B : 0.959

Caractéristiques de l'essai triaxial
Consolidation isotope par palliers
Dim. de l'échant.[mm]: Diam.:100.0 Haut.:97.094
Indice des vides: e=0.872
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
p'i=666.6 kPa  p'f=30.0 Kpa
qi=0 kPa  qf=43.1 kPa
Contrainte déviatorique

Cheminement des contraintes

Variation de la pression interstitielle

Essai triaxial Fort McMurray no.37
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 563 kPa $B = 0.988$

Caractéristiques de l'essai triaxial
Consolidation isotope
Dim. de l'échant.[mm]: Diam.:100.0 Haut.:99.623
indice des vides: $e = 0.783$
Type de chargement: Constant à 0.030 '/min.
Type de rupture: En tonneau
$p'' = 61.1$ kPa $p'' = 974$ Kpa
$q_i = 0$ kPa $q_f = 154.8$ kPa
Essai triaxial Fort McMurray n°38
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 628 kPa  B : 0.970

Caractéristiques de l'essai triaxial
Consolidation isotope
Dim. de l'échant.[mm]: Diam.:100.0  Haut.:97.592
Indice des vides: e=0.857
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
p'i=602.0 kPa  p'f=21.3 Kpa
qi=0 kPa  qf=28.2 kPa
Essai triaxial Fort McMurray no.49
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 450 kPa \( B : 0.996 \)

Caractéristiques de l'essai triaxial
Consolidation isotrope
Dim. de l'échantillon [mm]: Diam.:100.0 Haut.:99.599
Indice des vides: \( e = 0.724 \)
Type de chargement: Constant à 0.030 "/min.
Type de rupture: En tonneau
\( p' = 201.6 \) kPa \( p'_f = 414.3 \) Kpa
\( q_i = 0 \) kPa \( q'_f = 653.9 \) kPa
Cheminement des contraintes

Contrainte déviatorique

Variation de la pression interstitielle

Essai triaxial Fort McMurray no.50
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 447 kPa  $\bar{B}$ : 0.986

Caractéristiques de l'essai triaxial
Consolidation isotrope
Dim. de l'échant.[mm]: Diam.:100.0 Haut.:99.639
Indice des vides: e=0.760
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
$p'_{i}=204.0$ kPa  $p'_{f}=199.4$ Kpa
$q_{i}=0$ kPa  $q_{f}=308.7$ kPa
Contrainte déviatorique

Cheminement des contraintes

Variation de la pression interstitielle

Essai triaxial Fort McMurray no. 51
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 449 kPa  \( B : 0.994 \)

Caractéristiques de l'essai triaxial
Consolidation isotrope
Dim. de l'échant.[mm]: Diam.:100.0  Haut.:99.559
Indice des vides: \( e=0.744 \)
Type de chargement: Constant à 0.030°/min.
Type de rupture: En tonneau
\( p'i=203.0 \text{ kPa} \quad p'f=306.4 \text{ Kpa} \)
\( q_i=0 \text{ kPa} \quad q_f=479.9 \text{ kPa} \)
Essai triaxial Fort McMurray no.52
Échantillon non-gelé et remanié
Type de sol: Sable fin
Back pressure: 407 kPa \( \bar{B} : 0.995 \)

---

Caractéristiques de l'essai triaxial
Consolidation isotope
Dim. de l'échant.[mm]: Diam.:100.0 Haut.:99.612
Indice des vides: e=0.702
Type de chargement: Constant à 0.030 "/min.
Type de rupture: En tonneau
\( p'_{i}=202.8 \) kPa \( p'_{f}=727.6 \) Kpa
 qi=0 kPa \( q_f=1113.0 \) kPa
Cheminement des contraintes

Contrainte déviatorique

Variation de la pression interstitielle

Essai triaxial Fort McMurray no.53
Échantillon initialement gelé et remanié
Type de sol: Sable fin
Back pressure: 398 kPa  \( B : 0.981 \)

Caractéristiques de l'essai triaxial
Consolidation isotrope
Dim. de l'échant.[mm]: Diam.:100.0  Haut.:99.025
Indice des vides: e=0.765
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
\( p'i=204.1 \text{ kPa} \quad p'f=147.9 \text{ Kpa} \)
\( qi=0 \text{ kPa} \quad qf=227.4 \text{ kPa} \)
b) Undisturbed Phase I samples tested in triaxial compression
FIG. 3.7 Indice des vides lors de la progression du dégel de chaque essai

This void ratio increase is artificially created by coating the sample with ice.
### RÉSUMÉ DES ESSAIS SUR LES ÉCHANTILLONS INTACTS

<table>
<thead>
<tr>
<th>Essai no.</th>
<th>Échantillon no. profondeur(m)</th>
<th>Description du dégel</th>
<th>Type de dégel</th>
<th>Eau injectée dans l'échantillon durant le dégel</th>
<th>Théorique</th>
<th>Réalisée en laboratoire</th>
<th>ΔVr (ml)</th>
<th>ΔVr (ml)</th>
<th>e₀</th>
<th>Indices des vides après le dégel</th>
<th>Δeₐ=e₀-eₐ⁺</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM42</td>
<td>FS1-C8B (33.25)</td>
<td>Condition isotrope</td>
<td>20 6</td>
<td>32.7</td>
<td>3.6</td>
<td>0.11</td>
<td>0.729</td>
<td>0.736</td>
<td>0.729</td>
<td>0.000</td>
<td></td>
</tr>
<tr>
<td>FM43</td>
<td>FS1-C4 (30.60)</td>
<td>Condition isotrope</td>
<td>40 25</td>
<td>23.6</td>
<td>15.6</td>
<td>0.66</td>
<td>0.780</td>
<td>0.760</td>
<td>0.753</td>
<td>0.027</td>
<td></td>
</tr>
<tr>
<td>FM44</td>
<td>FS5-C5C (29.87)</td>
<td>Condition isotrope</td>
<td>41 29</td>
<td>22.5</td>
<td>22.0</td>
<td>0.98</td>
<td>0.710</td>
<td>0.694</td>
<td>0.693</td>
<td>0.017</td>
<td></td>
</tr>
<tr>
<td>FM45</td>
<td>FS3-C11A (33.15)</td>
<td>Condition isotrope</td>
<td>40 26</td>
<td>26.5</td>
<td>16.1</td>
<td>0.61</td>
<td>0.733</td>
<td>0.743</td>
<td>0.711</td>
<td>0.022</td>
<td></td>
</tr>
<tr>
<td>FM46</td>
<td>FS3-C10 (32.95)</td>
<td>Condition isotrope</td>
<td>41 27</td>
<td>26.1</td>
<td>19.2</td>
<td>0.74</td>
<td>0.719</td>
<td>0.730</td>
<td>0.699</td>
<td>0.020</td>
<td></td>
</tr>
<tr>
<td>FM47</td>
<td>FS1-C7 (32.50)</td>
<td>Condition isotrope</td>
<td>20 11</td>
<td>26.9</td>
<td>11.3</td>
<td>0.42</td>
<td>0.7425</td>
<td>0.709</td>
<td>0.707</td>
<td>0.035</td>
<td></td>
</tr>
<tr>
<td>FM48</td>
<td>FS3-C9 (32.43)</td>
<td>Condition isotrope</td>
<td>20 7</td>
<td>42.1</td>
<td>22.8</td>
<td>0.54</td>
<td>0.7298</td>
<td>0.730</td>
<td>0.720</td>
<td>0.010</td>
<td></td>
</tr>
<tr>
<td>FM55</td>
<td>FS4-C15B (36.87)</td>
<td>Condition isotrope</td>
<td>178 156</td>
<td>26.3</td>
<td>6.6</td>
<td>0.25</td>
<td>0.7559</td>
<td>0.760</td>
<td>0.720</td>
<td>0.036</td>
<td></td>
</tr>
</tbody>
</table>

** Cette quantité d'eau inclut le 8 mL d'eau ajoutée durant les opérations de mise en place de l'échantillon lors du montage.

### TABLEAU 3.1 Résumé des conditions de dégel de chaque échantillon intact
## RÉSUMÉ DES ESSAIS SUR LES ÉCHANTILLONS INTACTS

<table>
<thead>
<tr>
<th>Essai no.</th>
<th>Échantillon no. profondeur(m)</th>
<th>$e_s$</th>
<th>$\bar{B}$</th>
<th>Type de consolidation</th>
<th>Consolidation isotrope</th>
<th>Indices des vides après la consolidation isotrope</th>
<th>Consolidation anisotrope</th>
<th>Indices des vides après la consolidation anisotrope</th>
<th>$\Delta e = e_0 - e_+ $</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM42</td>
<td>FS1-C8B (33.25)</td>
<td>0.727</td>
<td>0.941</td>
<td>• Aux conditions naturelles</td>
<td>392 119</td>
<td>0.702</td>
<td>662 392 119</td>
<td>0.695 0.034</td>
<td></td>
</tr>
<tr>
<td>FM43</td>
<td>FS1-C4 (30.60)</td>
<td>0.749</td>
<td>0.717</td>
<td>• Aux conditions naturelles</td>
<td>319 86</td>
<td>0.714</td>
<td>551 319 86</td>
<td>0.699 0.081</td>
<td></td>
</tr>
<tr>
<td>FM44</td>
<td>FS5-C5C (29.87)</td>
<td>0.690</td>
<td>0.934</td>
<td>• Condi. effectives inférieures aux condi. in situ</td>
<td>649 544</td>
<td>0.682</td>
<td>879 649 544</td>
<td>0.674 0.036</td>
<td></td>
</tr>
<tr>
<td>FM45</td>
<td>FS3-C11A (33.15)</td>
<td>0.710</td>
<td>0.631</td>
<td>• Aux conditions naturelles</td>
<td>390 120</td>
<td>0.670</td>
<td>660 390 120</td>
<td>0.664 0.069</td>
<td></td>
</tr>
<tr>
<td>FM46</td>
<td>FS3-C10 (32.95)</td>
<td>0.697</td>
<td>0.917</td>
<td>• Aux conditions naturelles</td>
<td>389 120</td>
<td>0.675</td>
<td>659 389 120</td>
<td>0.671 0.048</td>
<td></td>
</tr>
<tr>
<td>FM47</td>
<td>FS1-C7 (32.50)</td>
<td>0.703</td>
<td>0.847</td>
<td>• Aux conditions naturelles</td>
<td>385 121</td>
<td>0.666</td>
<td>650 385 121</td>
<td>0.657 0.086</td>
<td></td>
</tr>
<tr>
<td>FM48</td>
<td>FS3-C9 (32.43)</td>
<td>0.719</td>
<td>0.880</td>
<td>• Aux conditions naturelles</td>
<td>390 123</td>
<td>0.699</td>
<td>655 390 123</td>
<td>0.693 0.037</td>
<td></td>
</tr>
<tr>
<td>FM55</td>
<td>FS4-C15B (36.87)</td>
<td>0.717</td>
<td>0.878</td>
<td>• Par paliers aux condi. effectives in situ</td>
<td>700 427</td>
<td>0.704</td>
<td>972 700 427</td>
<td>0.700 0.056</td>
<td></td>
</tr>
</tbody>
</table>

**TABLEAU 3.2** Résumé des conditions de consolidation de chaque échantillon intact
FIG. 3.8 Essais triaxiaux sur des échantillons intacts
FIG. 3.9 Enveloppe d'état critique du sable de Fort McMurray

\[
p'_{\text{normalisé}} = \frac{\left(\sigma'_1 + 2 \cdot \sigma'_3\right)}{\sigma'_c}
\]
Contrainte déviatorique

Contrainte déviatorique

Deformation axiale (%)

Deviateur (kPa)

Essai triaxial Fort McMurray no.42
Échantillon initialement gelé et non-remanié
Type de sol: Sable fin
Back pressure: 120 kPa  B : 0.941

Caractéristiques de l'essai triaxial
Consolidation anisotrope naturelle
Dim. de l'échantillon[mm]: Diam.:99.25  Haut.:110.079
Indice des vides: e=0.695
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
p'i=363.9 kPa  p'f=746.3 Kpa
q'i=274.1 kPa  qf=1184.7 kPa

Variation de la pression interstitielle

Variation de la pression interstitielle

Delta U (kPa)

Déformation axiale (%)

Cheminement des contraintes

Cheminement des contraintes

Deformation axiale (%)

Deviateur (kPa)

p' (kPa)
Essai triaxial Fort McMurray no. 43
Échantillon initialement gelé et non-remanié
Type de sol: Sable fin
Back pressure: 86.0 kPa  $B = 0.717$

Caractéristiques de l'essai triaxial
Consolidation anisotrope naturelle
Dim. de l'échantillon [mm]: Diam.: 93.9  Haut.: 91.6
Indice des vides: $e = 0.699$
Type de chargement: constant à 0.030 "/min.
Type de rupture: En tonneau
$p' = 311.7$ kPa  $p' = 657.6$ kPa
$q_i = 238.7$ kPa  $q_f = 1122.4$ kPa
Contrainte déviatorique

Variation de la pression interstitielle

Cheminement des contraintes

---

Essai triaxial Fort McMurray no.44
Échantillon initialement gelé et non-remanié
Type de sol: Sable fin
Back pressure: 544 kPa  $B : 0.931$

Caractéristiques de l'essai triaxial
Consolidation anisotrope
Dim. de l'échant. [mm]: Diam.:94.0  Haut.:91.8
Indice des vides: $e=0.674$
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
$p'_i=181.7$ kPa  $p'_f=798.8$ Kpa
$q_i=228.6$ kPa  $q_f=1393.2$ kPa
Contrainte déviatorique

Cheminement des contraintes

Variation de la pression interstitielle

Essai triaxial Fort McMurray no.45
Échantillon initialement gelé et non-remanié
Type de sol: Sable fin
Back pressure: 120 kPa \( B : 0.716 \)

Caractéristiques de l'essai triaxial
Consolidation anisotrope naturelle
Dim. de l'échant.[mm]: Diam.:93.6 Haut.:107.9
Indice des vides: \( e=0.664 \)
Type de chargement: Constant à 0.030 in/min.
Type de rupture: En tonneau
\( p'i=364.5 \text{ kPa} \)  \( p'f=711.0 \text{ Kpa} \)
\( q'i=283.9 \text{ kPa} \)  \( q'f=1129.0 \text{ kPa} \)
Contrainte déviatorique

Cheminement des contraintes

Variation de la pression interstitielle

Essai triaxial Fort McMurray no.46
Échantillon initialement gelé et non-remanié
Type de sol: Sable fin
Back pressure: 120 kPa  \( B : 0.920 \)

Caractéristiques de l'essai triaxial
Consolidation anisotrope naturelle
Dim. de l'échant.[mm]: Diam.:93.6 Haut.:108.2
Indice des vides: \( e=0.671 \)
Type de chargement: Constant à 0.030 "/min.
Type de rupture: En tonneau
\( p'_i=359.8 \text{ kPa} \quad p'_f=826.5 \text{ Kpa} \)
\( q_i=273.3 \text{ kPa} \quad q_f=1401.4 \text{ kPa} \)
Essai triaxial Fort McMurray no.47
Échantillon initialement gelé et non-remanié
Type de sol: Sable fin
Back pressure: 120 kPa $\bar{B} : 0.847$

Caractéristiques de l'essai triaxial
Consolidation anisotrope naturelle
Dim. de l'échant.[mm]: Diam.:93.5 Haut.:107.8
Indice des vides: $e=0.657$
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
$p_i'=356.6 \text{ kPa}$ $p_f'=788.0 \text{ Kpa}$
$q_i=276.7 \text{ kPa}$ $q_f=1270.6 \text{ kPa}$
Essai triaxial Fort McMurray no. 48
Échantillon initialement gelé et non-remanié
Type de sol: Sable fin
Back pressure: 120 kPa \(B: 0.880\)

Caractéristiques de l'essai triaxial
Consolidation anisotrope naturelle
Dim. de l'échant.[mm]: Diam.:94.9 Haut.:169.0
Indice des vides: \(e=0.6930\)
Type de chargement: Constant à 0.030 \%/min.
Type de rupture: En tonneau
\(p'_i=357.2\) kPa \(p'_f=819.7\) Kpa
\(q_i=267.7\) kPa \(q_f=1315.9\) kPa
Contrainte déviatorique

Contrainte de confinement

Essai triaxial Fort McMurray no.55
Échantillon initialement gelé et non-remanié
Type de sol: Sable fin
Back pressure: 427 kPa $B : 0.927$

Caractéristiques de l'essai triaxial
Consolidation anisotrope naturelle
Dim. de l'échant.[mm]: Diam.:95.7 Haut.:91.3
Indice des vides: $e=0.7004$
Type de chargement: Constant à 0.030 °/min.
Type de rupture: En tonneau
$p_i=360.4$ kPa  $p_f=1450.2$ Kpa
$q_i=262.0$ kPa  $q_f=2422.9$ kPa

Variation de la pression interstitielle

Cheminement des contraintes

$p'$ (kPa)
APPENDIX H

Phase I U. of A. Laboratory Testing Results
(Cunning, 1994; Ayoubian, 1996; Hofmann, 1996;
and cyclic triaxial tests)
1. Results from Cunning (1994)

Drained and undrained triaxial compression tests on reconstituted (moist tamped), isotropically consolidated samples of Phase I sand
<table>
<thead>
<tr>
<th>Test No.</th>
<th>p'c, (kPa)</th>
<th>e</th>
<th>Vs, (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS-339-T1-CU</td>
<td>48.0</td>
<td>0.936</td>
<td>108</td>
</tr>
<tr>
<td></td>
<td>94.8</td>
<td>0.919</td>
<td>134</td>
</tr>
<tr>
<td></td>
<td>154.0</td>
<td>0.904</td>
<td>158</td>
</tr>
<tr>
<td></td>
<td>197.0</td>
<td>0.896</td>
<td>175</td>
</tr>
<tr>
<td></td>
<td>250.0</td>
<td>0.888</td>
<td>189</td>
</tr>
<tr>
<td></td>
<td>297.0</td>
<td>0.883</td>
<td>202</td>
</tr>
<tr>
<td></td>
<td>348.0</td>
<td>0.877</td>
<td>217</td>
</tr>
<tr>
<td>SS-336-T2-CU</td>
<td>48.2</td>
<td>0.946</td>
<td>107</td>
</tr>
<tr>
<td></td>
<td>97.5</td>
<td>0.932</td>
<td>131</td>
</tr>
<tr>
<td></td>
<td>143.9</td>
<td>0.921</td>
<td>149</td>
</tr>
<tr>
<td></td>
<td>195.0</td>
<td>0.913</td>
<td>165</td>
</tr>
<tr>
<td></td>
<td>244.4</td>
<td>0.906</td>
<td>177</td>
</tr>
<tr>
<td></td>
<td>294.1</td>
<td>0.900</td>
<td>188</td>
</tr>
<tr>
<td></td>
<td>345.0</td>
<td>0.896</td>
<td>202</td>
</tr>
<tr>
<td>SS-249-T3-CU</td>
<td>48.7</td>
<td>0.934</td>
<td>102</td>
</tr>
<tr>
<td></td>
<td>92.8</td>
<td>0.917</td>
<td>128</td>
</tr>
<tr>
<td></td>
<td>142.9</td>
<td>0.904</td>
<td>146</td>
</tr>
<tr>
<td></td>
<td>191.4</td>
<td>0.895</td>
<td>161</td>
</tr>
<tr>
<td></td>
<td>246.3</td>
<td>0.887</td>
<td>198</td>
</tr>
<tr>
<td>SS-340-T4-CU</td>
<td>54.6</td>
<td>0.929</td>
<td>109</td>
</tr>
<tr>
<td></td>
<td>98.4</td>
<td>0.922</td>
<td>130</td>
</tr>
<tr>
<td></td>
<td>145.5</td>
<td>0.914</td>
<td>148</td>
</tr>
<tr>
<td></td>
<td>195.1</td>
<td>0.907</td>
<td>169</td>
</tr>
<tr>
<td></td>
<td>242.3</td>
<td>0.901</td>
<td>180</td>
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<tr>
<td></td>
<td>292.5</td>
<td>0.896</td>
<td>199</td>
</tr>
<tr>
<td></td>
<td>342.5</td>
<td>0.891</td>
<td>211</td>
</tr>
</tbody>
</table>

Table 4.4 Test data obtained during consolidation and at USS for Syncrude sand.
<table>
<thead>
<tr>
<th>Test No.</th>
<th>$p'$, (kPa)</th>
<th>$e$</th>
<th>$V_s$, (m/s)</th>
<th>$e'$</th>
<th>$V'_s$, (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS-54-T5-CD</td>
<td>51.0</td>
<td>0.909</td>
<td>124</td>
<td>148</td>
<td>182</td>
</tr>
<tr>
<td>SS-255-T6-CD</td>
<td>53.7</td>
<td>0.930</td>
<td>117</td>
<td>146</td>
<td>162</td>
</tr>
<tr>
<td></td>
<td>100.0</td>
<td>0.914</td>
<td>117</td>
<td>146</td>
<td>162</td>
</tr>
<tr>
<td></td>
<td>148.6</td>
<td>0.901</td>
<td>117</td>
<td>146</td>
<td>162</td>
</tr>
<tr>
<td></td>
<td>201.6</td>
<td>0.892</td>
<td>117</td>
<td>146</td>
<td>162</td>
</tr>
<tr>
<td></td>
<td>248.0</td>
<td>0.884</td>
<td>117</td>
<td>146</td>
<td>162</td>
</tr>
<tr>
<td>SS-350-T7-CD</td>
<td>49.1</td>
<td>0.935</td>
<td>106</td>
<td>155</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td>145.2</td>
<td>0.898</td>
<td>106</td>
<td>155</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td>246.1</td>
<td>0.883</td>
<td>106</td>
<td>155</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td>343.5</td>
<td>0.871</td>
<td>106</td>
<td>155</td>
<td>182</td>
</tr>
<tr>
<td>SS-437-T8(AP)-CU</td>
<td>54.0</td>
<td>0.792</td>
<td>137</td>
<td>32</td>
<td>215</td>
</tr>
<tr>
<td></td>
<td>145.8</td>
<td>0.774</td>
<td>137</td>
<td>32</td>
<td>215</td>
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<tr>
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<td>242.9</td>
<td>0.763</td>
<td>137</td>
<td>32</td>
<td>215</td>
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<td>342.3</td>
<td>0.755</td>
<td>137</td>
<td>32</td>
<td>215</td>
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<tr>
<td></td>
<td>440.0</td>
<td>0.749</td>
<td>137</td>
<td>32</td>
<td>215</td>
</tr>
<tr>
<td>SS-453-T9-CD</td>
<td>49.6</td>
<td>0.886</td>
<td>111</td>
<td>158</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td>141.5</td>
<td>0.856</td>
<td>111</td>
<td>158</td>
<td>182</td>
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<td>0.840</td>
<td>111</td>
<td>158</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td>340.9</td>
<td>0.829</td>
<td>111</td>
<td>158</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td>439.8</td>
<td>0.821</td>
<td>111</td>
<td>158</td>
<td>182</td>
</tr>
<tr>
<td>SS-352-T10-CU</td>
<td>52.2</td>
<td>0.924</td>
<td>118</td>
<td>158</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td>145.2</td>
<td>0.897</td>
<td>118</td>
<td>158</td>
<td>182</td>
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<tr>
<td></td>
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<td>0.881</td>
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<td>158</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td>342.3</td>
<td>0.870</td>
<td>118</td>
<td>158</td>
<td>182</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$(p'u'ss$, (kPa))</th>
<th>$(e'u'ss$, (kPa))</th>
<th>$(V's'u'ss$, (m/s))</th>
<th>$(\sigma'1)u'ss$, (kPa)</th>
<th>$(\sigma'3)u'ss$, (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS-339-T1-CU</td>
<td>11.1</td>
<td>0.877</td>
<td>79</td>
<td>21.6</td>
<td>5.8</td>
</tr>
<tr>
<td>SS-336-T2-CU</td>
<td>9.9</td>
<td>0.896</td>
<td>71</td>
<td>20.3</td>
<td>5.1</td>
</tr>
<tr>
<td>SS-249-T3-CU</td>
<td>8.4</td>
<td>0.887</td>
<td>65</td>
<td>16.6</td>
<td>4.4</td>
</tr>
<tr>
<td>SS-340-T4-CU</td>
<td>6.8</td>
<td>0.891</td>
<td>-</td>
<td>14.4</td>
<td>2.9</td>
</tr>
<tr>
<td>SS-54-T3-CD</td>
<td>102.5</td>
<td>0.808</td>
<td>160</td>
<td>192.8</td>
<td>57.3</td>
</tr>
<tr>
<td>SS-255-T6-CD</td>
<td>431.3</td>
<td>0.789</td>
<td>243</td>
<td>796.5</td>
<td>248.7</td>
</tr>
<tr>
<td>SS-350-T7-CD</td>
<td>607.9</td>
<td>0.779</td>
<td>236</td>
<td>1138.9</td>
<td>342.3</td>
</tr>
<tr>
<td>SS-437-T8(AP)-CU</td>
<td>298.5</td>
<td>0.749</td>
<td>221</td>
<td>571.0</td>
<td>162.2</td>
</tr>
<tr>
<td>SS-453-T9-CD</td>
<td>779.0</td>
<td>0.724</td>
<td>343</td>
<td>1454.6</td>
<td>441.3</td>
</tr>
<tr>
<td>SS-352-T10-CU</td>
<td>31.6</td>
<td>0.870</td>
<td>94</td>
<td>66.2</td>
<td>13.9</td>
</tr>
</tbody>
</table>

Table 4.4 Continued.
Figure 4.28 Syncrude sand drained test results in a). $q$ against axial strain and b). change in volume against axial strain.
Figure 4.27 Syncrude sand undrained test results in a). $q$ against axial strain and b). change in pore pressure against axial strain.
Figure 4.29 Normalized stress paths for test on Syncrude sand. a). shows undrained test with collapse line, b). shows drained tests, and a expanded view of undrained tests collapsing to USS.
2. Results from Ayoubian (1996)
a) Reconstituted (moist tamped), anisotropically consolidated Phase I samples tested in triaxial compression and extension
FIGURE 4.10 Consolidation Curves and Ultimate Steady States for AC Tests on Syncrude Sand
FIGURE 4.11 Stress Paths for AC Tests on Syncrude Sand
FIGURE 4.12 Syncrude Sand AC Test Results
a) Deviator Stress (q) vs Axial Strain
b) Change in Pore Pressure vs Axial Strain
b) One undisturbed Phase I sample tested in triaxial compression
FIGURE 4.16 In-situ and Ultimate Steady State for AC Test on Undisturbed Sample of Syncrude Sand
FIGURE 4.17 Stress Path for AC Test on Undisturbed Sample of Syncrude Sand (SSF)
FIGURE 4.18 Syncrude Sand AC Test Results
(Undisturbed sample)

a) Deviator Stress ($q$) vs Axial Strain
b) Pore Pressure vs Axial Strain
3. Results from Hofmann (1997)
a) Undisturbed Phase I samples tested in triaxial compression and extension
Pore Pressure Response

Triaxial Compression Test on Undisturbed Phase I Syncrude Sand Specimen FS5C14

Initial Frozen Void Ratio: 0.746  
Sr = 96%  
B bar = 0.372

Thawing Conditions: Unidirectional thawing from base of specimen under in-situ stresses

Stresses During Thawing and Consolidation:

- $\sigma_{\text{axial}} = 602$ kPa  
- $q_s = 234$ kPa  
- $\sigma_0 = 368$ kPa  
- $p'_s = 312$ kPa  
- $U = 133$ kPa  
- $K_c = 0.5$

Void Ratio after Thawing and Consolidation: 0.737

Steady State Condition Reached:  
$q_{ss} = 1458$ kPa  
$p'_{ss} = 859$ kPa
Stress Strain Curve

q-p' Diagram

Pore Pressure Response

Triaxial Compression Test on Undisturbed Phase I Syncrude Sand Specimen FS4C14A

Initial Frozen Void Ratio: 0.847 Sr = 85%
B bar = 1

Thawing Conditions: Unidirectional thawing from base of specimen under in-situ stresses

Stresses During Thawing and Consolidation:

\[ \sigma_{axial} = 627 \text{ kPa} \quad q_u = 240 \text{ kPa} \]
\[ \sigma_{cax} = 387 \text{ kPa} \quad p'_{o} = 320 \text{ kPa} \]
\[ U = 147 \text{ kPa} \]
\[ K_s = 0.5 \]

Void Ratio after Thawing and Consolidation: 0.834

Steady State Condition Reached:
\[ q_{ss} = 934 \text{ kPa} \]
\[ p'_{ss} = 632 \text{ kPa} \]

9/9/97
Stress Strain Curve

Deviant Stress, q (kPa)

Axial Strain (%)

Stress Strain Curve

Deviant Stress, q (kPa)

Axial Strain (%)

q-p' Diagram

Deviant stress, q (kPa)

Mean Principal stress, p' (kPa)

Pore Pressure Response

Change in Pore Pressure (kPa)

Axial Strain (%)

Triaxial Compression Test on Undisturbed Phase I Syncrude Sand Specimen FS3C17B

Initial Frozen Void Ratio: 0.764 Sr = 100% (back saturated)
B bar = 1

Thawing Conditions: Unidirectional thawing from base of specimen under in-situ stresses

Stresses During Thawing and Consolidation:

\[
\begin{align*}
\sigma_{\text{axial}} &= 658 \text{ kPa} \\
\sigma_{\text{tan}} &= 411 \text{ kPa} \\
U &= 163 \text{ kPa} \\
K_s &= 0.6 \\
p' &= 330 \text{ kPa} \\
qu &= 2014 \text{ kPa} \\
p'_{\text{es}} &= 1350 \text{ kPa}
\end{align*}
\]

Void Ratio after Thawing and Consolidation: 0.734

Steady State Condition Reached:

q = 2014 kPa

9/9/97
Stress Strain Curve

Pore Pressure Response

q-p’ Diagram

Triaxial Extension Test on Undisturbed Phase I Syncrude Sand Specimen FS5C10A

Initial Frozen Void Ratio: 0.728

\[ Sr = 87.31\% \]

Thawing Conditions: Unidirectional thawing from base of specimen under In-situ stresses

Stresses During Thawing and Consolidation:

\[ \sigma_{\text{axial}} = 562 \text{ kPa} \quad q = 225 \text{ kPa} \]
\[ \sigma_{\text{int}} = 337 \text{ kPa} \quad p' = 300 \text{ kPa} \]
\[ U = 112 \text{ kPa} \]
\[ K_s = 0.5 \]

Void Ratio after Thawing and Consolidation: 0.704

Steady State Condition Reached:

\[ q_{ss} = -250 \text{ kPa} \]
\[ p'_{ss} = 230 \text{ kPa} \]
Triaxial Extension Test on Undisturbed Phase I Syncrude Sand Specimen FS4C162

Initial Frozen Void Ratio: 0.729  \( Sr = 97.46\% \)

B bar = 0.55

Thawing Conditions: Unidirectional thawing from base of specimen under in-situ stresses

Stresses During Thawing and Consolidation:

\[
\begin{align*}
\sigma_{axial} &= 655 \ \text{kPa} \\
\sigma_{rad} &= 408 \ \text{kPa} \\
q &= 246 \ \text{kPa} \\
p' &= 329 \ \text{kPa} \\
U &= 161 \ \text{kPa} \\
K_s &= 0.5
\end{align*}
\]

Void Ratio after Thawing and Consolidation: 0.612

Steady State Condition Reached:

\[
\begin{align*}
q' &= -283 \ \text{kPa} \\
p'' &= 334 \ \text{kPa}
\end{align*}
\]
b) One Phase I sample tested in triaxial compression that was reconstituted (isotropically consolidated) from a previously tested undisturbed sample
Stress Strain Curve

q-p' Diagram

Pore Pressure Response

Triaxial Compression Test on Reconstituted Syncrude Sand Specimen FS5C14R
(Undisturbed sample FS5C14 was tested and then reconstituted to form this sample)

Initial Void Ratio: 0.702
Sr = 100%
B bar = 1

Stresses During Consolidation:

\[ \sigma_{axial} = 315 \text{ kPa} \]
\[ q_o = 0 \text{ kPa} \]
\[ \sigma_{cap} = 315 \text{ kPa} \]
\[ p'_{o} = 35 \text{ kPa} \]
\[ U = 280 \text{ kPa} \]
\[ K_o = 0.6 \]

Void Ratio after Consolidation: 0.700

Steady State Condition Reached:

\[ q_{ss} = 1872 \text{ kPa} \]
\[ p'_{ss} = 979 \text{ kPa} \]
4. Cyclic triaxial tests on undisturbed Phase I samples
FORMULAE USED IN CALCULATION SHEETS

Measurements:

\[ \begin{align*}
H & \quad \text{height of frozen sample} \\
D & \quad \text{diameter of frozen sample} \\
M & \quad \text{mass of frozen sample} \\
V & \quad \text{volume of frozen sample} \\
w_{\text{ice}} & \quad \text{initial ice content of sample} \\
& \quad (= \text{ice content of chunk trimmed from end of sample}) \\
M_s & \quad \text{mass of dry soil} \\
w_f & \quad \text{final water content of whole sample after testing is completed} \\
G_s & \quad \text{specific gravity of sand} = 2.66 \text{ for Phase I sand} \\
G_{\text{ice}} & \quad \text{specific gravity of ice} = 0.917 \\
\Delta H & \quad \text{change in height of sample during thaw} \\
\Delta V & \quad \text{change in volume of sample during consolidation} \\
& \quad \text{(determined by multiplying the volumetric strain measured by the computer by V)}
\end{align*} \]

Calculations:

\[ \begin{align*}
\rho_{\text{frozen}} & \quad \text{frozen density} \\
\rho_{\text{dry}} & \quad \text{dry density} \\
e_i & \quad \text{in-situ void ratio} \\
S_r & \quad \text{in-situ degree of saturation} \\
e_{\text{thaw}} & \quad \text{void ratio after thaw} \\
e_c & \quad \text{void ratio after thaw \& consolidation} \\
(S_r)_{\text{lab}} & \quad \text{laboratory degree of saturation} \\
& \quad \text{(during undrained cyclic loading)} \\
M/V & \quad \rho_{\text{frozen}}/(1 + w_{\text{ice}}) \\
& \quad G_s/p_d - 1 \\
& \quad (w_{\text{ice}} G_s)/(e_i G_{\text{ice}}) \\
& \quad e_i - [(\Delta H)(\pi/4)(D^2)]/(M_s G_s) \\
& \quad e_{\text{thaw}} - (1 + e_{\text{thaw}})(\Delta V/V) \\
& \quad (w_f G_s)/(e_c)
\end{align*} \]
Canlex Tests

(Cyclic Consolidated - Undrained Test)

Sample Location: FS4 C3A 29.73 - 29.86 m

Sample Dimensions

Sample name: FS4 C3A 29.73 - 29.86 m
Test Name: Consolidated - Undrained Test
Date: Sept. 11 / 97

Initial Values:

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Height</th>
<th>Mass</th>
<th>Initial Ice Content</th>
<th>Final Water Content</th>
<th>Dry Mass of Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>63.89 mm</td>
<td>130.44 mm</td>
<td>792.33 g</td>
<td>23.49 % by mass</td>
<td>20.5 % by mass</td>
<td>660.5 g</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Volume</th>
<th>Specific gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>418.183 cm³</td>
<td>2.66</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wet Density</th>
<th>Dry Density</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.895 g/cm³</td>
<td>1.534 g/cm³</td>
<td>0.734</td>
</tr>
</tbody>
</table>

Void ratio changes during phases of the test procedure:

<table>
<thead>
<tr>
<th>Total Volume</th>
<th>Void Ratio</th>
<th>Δe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ht. change (mm)</td>
<td>Volume change (cc)</td>
<td></td>
</tr>
<tr>
<td>Initial</td>
<td>0.00</td>
<td>0.734</td>
</tr>
<tr>
<td>After Thaw (decrease)</td>
<td>0.53</td>
<td>0.727</td>
</tr>
<tr>
<td>After Consolidation</td>
<td>7.1</td>
<td>0.704</td>
</tr>
<tr>
<td>End of Shear Test</td>
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<td>0.704</td>
</tr>
</tbody>
</table>

In-situ Sr (%): 92.9
Lab Sr (%): 77.4
Canlex Tests (Cyclic Consolidated - Undrained Test)

Sample Location: FS5 C15A 34.86 - 35.0 m

Sample Dimensions

Sample name: FS5 C15A 34.86 - 35.0 m
Test Name: Cyclic Consolidated - Undrained Test
Date: Sept. 23 / 97

Initial Values:

| Diameter:  | 64.37 mm |
| Height:    | 122.90 mm |
| Mass:      | 745.62 g  |
| Initial Ice content: | 23.01 % by mass |
| Final water content: | 22.0 % by mass |
| Dry mass of sample: | 606.0 g |

Volume: 399.953 cm³, Specific gravity: 2.66
Wet Density: 1.864 g/cm³, Dry Density: 1.516 g/cm³, Void Ratio: 0.755

Void ratio changes during phases of the test procedure.

<table>
<thead>
<tr>
<th>Total Ht. change (mm)</th>
<th>Volume Change (cc)</th>
<th>Void Ratio</th>
<th>Δe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td>0.00</td>
<td>0.755</td>
<td></td>
</tr>
<tr>
<td>After Thaw (decrease)</td>
<td>4.50</td>
<td>0.691</td>
<td>0.064</td>
</tr>
<tr>
<td>After Consolidation</td>
<td>13.5</td>
<td>0.632</td>
<td>0.059</td>
</tr>
<tr>
<td>End of Shear Test</td>
<td></td>
<td>0.632</td>
<td></td>
</tr>
</tbody>
</table>

In-situ S, (%) | Lab S, (%)
88.4          | 92.6
Canlex Tests

(Cyclic Consolidated - Undrained Test)

Sample Location: FS1 C5 (30.86 - 30.99 m)

Sample Dimensions

Sample name: FS1 C5 (30.86 - 30.99 m)
Test Name: Cyclic Consolidated - Undrained Test
Date: Sept. 25/97

*Note: the sample was isotropically consolidated to approximately the same mean normal effective stress as in the field

Initial Values:

| Diameter | 63.08 mm |
| Height | 132.67 mm |
| Mass | 764.30 g |
| Initial ice content | 23.80 % by mass |
| Final water content | 26.5 % by mass |
| Dry mass of sample | 603.08 g |
| Volume | 414.616 cm³ |
| Specific gravity | 2.66 |
| Wet Density | 1.843 g/cm³ |
| Dry Density | 1.489 g/cm³ |
| Void Ratio | 0.786 |

Void ratio changes during phases of the test procedure.

<table>
<thead>
<tr>
<th>Ht. change</th>
<th>Volume Change</th>
<th>Void Ratio</th>
<th>Δe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td>0.00</td>
<td>0.786</td>
<td></td>
</tr>
<tr>
<td>After Thaw</td>
<td>(decrease)</td>
<td>0.00</td>
<td>0.786</td>
</tr>
<tr>
<td>After Consolidation</td>
<td>11.4</td>
<td>0.737</td>
<td>0.049</td>
</tr>
<tr>
<td>End of Shear Test</td>
<td>0</td>
<td>0.737</td>
<td></td>
</tr>
</tbody>
</table>

In-situ Sr (%) | Lab Sr (%) | 87.8 | 95.6
Canlex Tests

(Cyclic Consolidated - Undrained Test)

Sample Location: FS5 C16B 35.59 - 35.81 m

* Note: the sample was isotropically consolidated to approximately the same mean normal effective stress as in the field

Sample Dimensions

Sample name: FS5 C16B 35.59 - 35.81 m
Test Name: Cyclic Consolidated - Undrained Test
Date: Sept. 30 / 97

Initial Values:

<table>
<thead>
<tr>
<th>Diameter</th>
<th>63.57 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height</td>
<td>132.13 mm</td>
</tr>
<tr>
<td>Mass</td>
<td>762.80 g</td>
</tr>
<tr>
<td>Initial Ice content</td>
<td>22.57 % by mass</td>
</tr>
<tr>
<td>Final water content</td>
<td>23.2 % by mass</td>
</tr>
<tr>
<td>Dry mass of sample</td>
<td>613.8 g</td>
</tr>
</tbody>
</table>

| Volume          | 419.368 cm³ |
| Wet Density     | 1.819 g/cm³ |
| Dry Density     | 1.484 g/cm³ |
| Void Ratio      | 0.792       |
| Specific gravity| 2.66        |

Void ratio changes during phases of the test procedure.

<table>
<thead>
<tr>
<th></th>
<th>Total Ht. change (mm)</th>
<th>Volume Change (cc)</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td>0.00</td>
<td>0.792</td>
<td></td>
</tr>
<tr>
<td>After Thaw: (decrease)</td>
<td>0.05</td>
<td>0.792</td>
<td>0.001</td>
</tr>
<tr>
<td>After Consolidation:</td>
<td>14.0</td>
<td>0.732</td>
<td>0.060</td>
</tr>
<tr>
<td>End of Shear Test:</td>
<td></td>
<td></td>
<td>0.732</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>In-situ S, (%)</th>
<th>Lab S, (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>82.6</td>
<td>84.3</td>
</tr>
</tbody>
</table>
Canlex Tests

( Cyclic Consolidated - Undrained Test )

Sample Location: FS4 C6A 30.71 - 30.93 m

* Note: the sample was isotropically consolidated to approximately the same mean normal effective stress as in the field

Sample Dimensions:

Sample name: FS4 C6A 30.71 - 30.93 m
Test Name: Cyclic Consolidated - Undrained Test
Date: Oct. 1 / 97

Initial Values:

| Diameter: | 63.64 mm |
| Height: | 131.32 mm |
| Mass: | 779.37 g |
| Initial ice content: | 24.75 % by mass |
| Final water content: | 27.0 % by mass |
| Dry mass of sample: | 613.0 g |

| Volume: | 417.716 cm³ |
| Specific gravity: | 2.66 |
| Wet Density: | 1.866 g/cm³ |
| Dry Density: | 1.496 g/cm³ |
| Void Ratio: | 0.779 |

Void ratio changes during phases of the test procedure:

<table>
<thead>
<tr>
<th>Total Ht. change (mm)</th>
<th>Total Volume change (cc)</th>
<th>Void Ratio Δe</th>
<th>In-situ Sr (%)</th>
<th>Lab Sr (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td>0.00</td>
<td>0.779</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After Thaw :</td>
<td>(decrease) 1.39</td>
<td>0.760</td>
<td>0.019</td>
<td>92.2</td>
</tr>
<tr>
<td>After Consolidation :</td>
<td>12.3</td>
<td>0.708</td>
<td>0.052</td>
<td>101.5</td>
</tr>
<tr>
<td>End of Shear Test :</td>
<td></td>
<td>0.708</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
\[ \text{efield} = \text{elab} \]

- Poor quality
- \( U_{\text{ofA}} \)
SAMPLE FS5 C15A

Number of Cycles, N

Deviator Stress (MPa)

Axial Strain (%)

Pore Pressure (kPa)

Effective Confining Pressure, $\sigma_3$ (kPa)
SAMPLE FS5 C15A

Axial Strain (%) vs. Deviator Stress (kPa)

p' (kPa) vs. q (kPa)
SAMPLE FS5 C16B
Summary of cyclic triaxial testing results for Phase I undisturbed samples

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sample No.</th>
<th>$e_c$</th>
<th>$\sigma'_3$, $\sigma'_1$ (kPa)</th>
<th>$S_r$ (%)</th>
<th>Triaxial CSR $\sigma'_d/(2\sigma'_3)$</th>
<th>N</th>
<th>$M^*$</th>
<th>Correction $CRR_M$</th>
<th>$CRR_{M=7.5}$ (M=7.5)</th>
<th>$CRR_{ss}$ (M=7.5) = 0.7$CRR_{tx}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>.29.78</td>
<td>FS4 C3A</td>
<td>0.704</td>
<td>311</td>
<td>77.4</td>
<td>0.32</td>
<td>55</td>
<td>9.33</td>
<td>0.78</td>
<td>0.410</td>
<td>0.287</td>
</tr>
<tr>
<td>34.93</td>
<td>FS5 C15A **</td>
<td>0.632</td>
<td>346</td>
<td>92.6</td>
<td>0.361</td>
<td>1</td>
<td>4.94</td>
<td>1.65</td>
<td>0.219</td>
<td>0.153</td>
</tr>
<tr>
<td>35.065</td>
<td>FS1 C5</td>
<td>0.737</td>
<td>324</td>
<td>95.6</td>
<td>0.185</td>
<td>11</td>
<td>6.93</td>
<td>1.10</td>
<td>0.169</td>
<td>0.118</td>
</tr>
<tr>
<td>35.75</td>
<td>FS5 C16B</td>
<td>0.732</td>
<td>350</td>
<td>84.2</td>
<td>0.36</td>
<td>22</td>
<td>8.24</td>
<td>0.90</td>
<td>0.401</td>
<td>0.281</td>
</tr>
<tr>
<td>30.82</td>
<td>FS4 C6A</td>
<td>0.708</td>
<td>325</td>
<td>101.4</td>
<td>0.231</td>
<td>76</td>
<td>9.78</td>
<td>0.74</td>
<td>0.312</td>
<td>0.218</td>
</tr>
</tbody>
</table>

Notes: *

To calculate the equivalent earthquake magnitude, $M$, from the number of cycles of uniform loading, $N$, the following equations were used: for $N \leq 35$, $M = -0.0038 N^2 + 0.2442 N + 4.7034$ (the equation given in the Introductory Data Review Report); for $N > 30$, $M = 1.3831 \ln(N) + 3.7876$ (a different formula which gives better results for high values of $N$). Note that for $2 < N < 30$, the two formulae give similar values of $M$ for a given value of $N$.

** Poor quality test; results likely unreliable.
Value beside each dot indicates the degree of saturation of the sample in the laboratory during the cyclic test.

Peak cyclic pore pressure ratio = 100% (N=15 cycles) (Seed et al., 1985)

\[ e_{\text{max}} = \]}

\[ e_{\text{min}} = 0.668 \]
Values beside each dot indicates the void ratio of the sample.
APPENDIX I

Industry Laboratory Testing Results

1. Klohn-Crippen Results

2. C-CORE Results
1. Klohn-Crippen Laboratory Testing Results
   (Plewes, 1995)
2. C-CORE Laboratory Testing Results
   (C-CORE, 1993)
Undrained Triaxial Extension Test
Confining stress 50kPa, Dr=37.7%, B=0.95
Undrained Triaxial Extension Test
Confining stress 100kPa, Dr=35.1%, B=0.97
Undrained Triaxial Extension Test

Confining stress 200kPa, Dr=43.7%, B=0.92
Undrained Triaxial Extension Test
Confining stress 400kPa, Dr=42.6%, B=0.98
Undrained Triaxial Compression Test
Confining stress 100kPa, Dr=39.8%, B=0.91
Undrained Triaxial Compression Test
Confining stress 200kPa, Dr=40.5%, B=0.93
Undrained Triaxial Compression Test
Confining stress 400kPa, Dr=43.0%, B=0.93
Drained Triaxial Compression Test
Confining stress 50kPa, Dr=38.1%
Drained Triaxial Compression Test
Confining stress 100kPa, Dr=36.7%
Drained Triaxial Compression Test
Confining stress 200kPa, Dr=36.5%
Volumetric compression of Canlex sand (Dr=39.6%)

Confining pressure (kPa)

Mean effective principal stress, $p$ (kPa)

Deviator stress, $q$ (kPa)

Oil Sand Volumetric Compression
Summary of Triaxial Test Stress Paths
APPENDIX J

Phase III Site SPT Results
(Iravani et al., 1995)
### Table 2.2.1 - SPT Locations

<table>
<thead>
<tr>
<th>SPT</th>
<th>Northing</th>
<th>Easting</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT 1</td>
<td>58706</td>
<td>50115</td>
</tr>
<tr>
<td>SPT 2</td>
<td>58701.25</td>
<td>50130</td>
</tr>
<tr>
<td>SPT 3</td>
<td>58711</td>
<td>50120</td>
</tr>
<tr>
<td>SPT 4</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

### Table 2.2.2.A - SPT Energy Measurements

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Depth (m)</th>
<th>Blow Count</th>
<th>N\text{valid}</th>
<th>Min. ER. (%)</th>
<th>Max. ER. (%)</th>
<th>Avg. ER. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.01</td>
<td>2/2/1</td>
<td>3</td>
<td>40.9</td>
<td>47.0</td>
<td>43.3</td>
</tr>
<tr>
<td>2</td>
<td>8.53</td>
<td>1/2/1</td>
<td>3</td>
<td>43.6</td>
<td>55.0</td>
<td>49.3</td>
</tr>
<tr>
<td>3</td>
<td>10.06</td>
<td>1/1/1</td>
<td>2</td>
<td>56.3</td>
<td>56.3</td>
<td>56.3</td>
</tr>
</tbody>
</table>

### Table 2.2.2.B - Results of SPT 1

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Depth (m)</th>
<th>RL\text{\textcircled{\small o}}</th>
<th>C_r</th>
<th>C_N</th>
<th>Blow Count</th>
<th>N\text{SPT}</th>
<th>(N_1)_{60}</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT 1-1</td>
<td>2.44</td>
<td>3.89</td>
<td>0.75</td>
<td>2.13</td>
<td>4/3/2</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>SPT 1-2</td>
<td>3.81</td>
<td>5.41</td>
<td>0.85</td>
<td>1.70</td>
<td>1/0/0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SPT 1-3</td>
<td>4.27</td>
<td>5.41</td>
<td>0.85</td>
<td>1.61</td>
<td>4/3/2</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>SPT 1-4</td>
<td>5.49</td>
<td>6.93</td>
<td>0.85</td>
<td>1.42</td>
<td>1/2/2</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>SPT 1-5</td>
<td>6.25</td>
<td>8.46</td>
<td>0.95</td>
<td>1.33</td>
<td>1/2/2</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>SPT 1-6</td>
<td>7.01</td>
<td>8.46</td>
<td>0.95</td>
<td>1.26</td>
<td>1/0/0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SPT 1-7</td>
<td>7.77</td>
<td>9.98</td>
<td>0.95</td>
<td>1.19</td>
<td>2/1/1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>SPT 1-8</td>
<td>8.53</td>
<td>9.98</td>
<td>0.95</td>
<td>1.14</td>
<td>1/1/2</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>SPT 1-9</td>
<td>9.68</td>
<td>11.51</td>
<td>1.00</td>
<td>1.07</td>
<td>1/0/1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>SPT 1-10</td>
<td>10.82</td>
<td>13.03</td>
<td>1.00</td>
<td>1.01</td>
<td>7/13/22</td>
<td>35</td>
<td>33</td>
</tr>
</tbody>
</table>

\textcircled{\small o} Rod Length
* Rod sinking under self weight.
** Test was performed in clay and clayshale formation.
**Table 2.2.2.C - Results of SPT 2**

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Depth (m)</th>
<th>RL °(m)</th>
<th>Cr</th>
<th>Cn</th>
<th>Blow Count</th>
<th>NSPT</th>
<th>(N₁)₆₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT 2-1</td>
<td>2.00</td>
<td>3.89</td>
<td>0.75</td>
<td>2.35</td>
<td>8/1/1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>SPT 2-2</td>
<td>2.75</td>
<td>5.41</td>
<td>0.85</td>
<td>2.00</td>
<td>8/1/1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>SPT 2-3</td>
<td>3.50</td>
<td>5.41</td>
<td>0.85</td>
<td>1.78</td>
<td>3/1/1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>SPT 2-4</td>
<td>4.25</td>
<td>6.93</td>
<td>0.85</td>
<td>1.61</td>
<td>3/1/1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>SPT 2-5</td>
<td>5.00</td>
<td>6.93</td>
<td>0.85</td>
<td>1.49</td>
<td>2/1/2</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>SPT 2-6</td>
<td>5.75</td>
<td>8.46</td>
<td>0.95</td>
<td>1.39</td>
<td>2/1/1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>SPT 2-7</td>
<td>6.50</td>
<td>8.46</td>
<td>0.95</td>
<td>1.30</td>
<td>2/2/1</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>SPT 2-8</td>
<td>7.25</td>
<td>9.98</td>
<td>0.95</td>
<td>1.23</td>
<td>2/1/1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>SPT 2-9</td>
<td>8.00</td>
<td>9.98</td>
<td>0.95</td>
<td>1.18</td>
<td>2/2/4</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>SPT 2-10</td>
<td>8.75</td>
<td>11.51</td>
<td>1.00</td>
<td>1.12</td>
<td>3/1/1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>SPT 2-11</td>
<td>9.50</td>
<td>11.51</td>
<td>1.00</td>
<td>1.08</td>
<td>4/2/1</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>SPT 2-12</td>
<td>10.25</td>
<td>13.03</td>
<td>1.00</td>
<td>1.04</td>
<td>2/1/0</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>SPT 2-13</td>
<td>11.00</td>
<td>13.03</td>
<td>1.00</td>
<td>1.00</td>
<td>1/2/4</td>
<td>6</td>
<td>6</td>
</tr>
</tbody>
</table>

⊗ Rod Length

+ Sand blockage in casing.

**Table 2.2.2.D - Results of SPT 3**

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Depth (m)</th>
<th>RL °(m)</th>
<th>Cr</th>
<th>Cn</th>
<th>Blow Count</th>
<th>NSPT</th>
<th>(N₁)₆₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT 3-1</td>
<td>2.44</td>
<td>3.89</td>
<td>0.75</td>
<td>2.13</td>
<td>6/5/4</td>
<td>9</td>
<td>13</td>
</tr>
<tr>
<td>SPT 3-2</td>
<td>4.11</td>
<td>5.41</td>
<td>0.85</td>
<td>1.64</td>
<td>0/1/1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>SPT 3-3</td>
<td>5.49</td>
<td>6.93</td>
<td>0.85</td>
<td>1.42</td>
<td>1/1/1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>SPT 3-4</td>
<td>7.01</td>
<td>8.46</td>
<td>0.95</td>
<td>1.26</td>
<td>1/2/2</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>SPT 3-5</td>
<td>8.53</td>
<td>9.98</td>
<td>0.95</td>
<td>1.14</td>
<td>1/1/1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>SPT 3-6</td>
<td>10.13</td>
<td>11.51</td>
<td>1.00</td>
<td>1.04</td>
<td>1/0/1</td>
<td>1</td>
<td>1</td>
</tr>
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<td>SPT 3-7**</td>
<td>11.58</td>
<td>13.03</td>
<td>1.00</td>
<td>0.98</td>
<td>1/3/4</td>
<td>7</td>
<td>6</td>
</tr>
</tbody>
</table>

Notes:
- Test was performed every 1.5 m (5 feet) to minimize any possible disturbance.
- The bore hole remained open up to 11.60m depth.
- ⊗ Rod Length
- ** Test was performed in clay and clayshale formation.
Figure 2.2.2.A - CANLEX Phase III, SPT Results
Figure 2.2.2.B - Grain Size Analysis (SPT 1 Samples)
Figure 2.2.2.C - Grain Size Analysis (SPT 2 Samples)
Figure 2.2.2.D - Grain Size Analysis (SPT 3 Samples)
Figure 2.2.2.E - Grain Size Profile

Test Depth (m)

Fines (% Passing #200 Sieve)
APPENDIX K

Phase III Site CPT Results
(Iravani et al., 1995; after ConeTec, 1995)

N.B. An extensive CPT investigation was carried out across the entire Phase III site. The CPT results presented here are those from or close to the detailed site characterization area.
Table 1.2 - List of *In-Situ* Testing

<table>
<thead>
<tr>
<th>U. of A. Symbol</th>
<th>Syncrude Symbol</th>
<th>Test Type</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT 1</td>
<td>GTTH9516C -21</td>
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</tr>
<tr>
<td>CPT 2</td>
<td>GTTH9516C -17</td>
<td>CPTU</td>
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</tr>
<tr>
<td>CPT 4</td>
<td>GTTH9516C -10</td>
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</tr>
<tr>
<td>CPT 5</td>
<td>GTTH9516C -20</td>
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<tr>
<td>CPT 6</td>
<td>GTTH9516C -16</td>
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<td>CPT 9</td>
<td>GTTH9516C -19</td>
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</tr>
<tr>
<td>CPT 10</td>
<td>GTTH9516C -15</td>
<td>CPTU</td>
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</tr>
<tr>
<td>CPT 11</td>
<td>GTTH9516C -12</td>
<td>SCPTU</td>
<td></td>
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<tr>
<td>CPT 12</td>
<td>GTTH9516C -8</td>
<td>CPTU</td>
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<tr>
<td>CPT 13</td>
<td>GTTH9516C -18</td>
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<td>CPT 15</td>
<td>GTTH9516C -11</td>
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<td>CPT 16</td>
<td>GTTH9516C -7</td>
<td>CPTU</td>
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<td>GTTH9516C -22</td>
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<td>GTTH9516C -23</td>
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<td>CPT 19</td>
<td>GTTH9516C -24</td>
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<tr>
<td>CPT 20</td>
<td>GTTH9516C -25</td>
<td>CPTU</td>
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<td>GTTH9516C -26</td>
<td>CPTU</td>
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<tr>
<td>CPT 26</td>
<td>GTTH9516C -31</td>
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<td></td>
</tr>
<tr>
<td>CPT 27</td>
<td>GTTH9516C -32</td>
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<td></td>
</tr>
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<td>GTTH9516C -75</td>
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<td>CPT 29</td>
<td>GTTH9516C -76</td>
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<td>After Embankment Construction</td>
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<td>CPT 32</td>
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<td>CPT 33</td>
<td>GTTH9516C -80</td>
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<td>CPT 34</td>
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<td>CPT 35</td>
<td>GTTH9516C -82</td>
<td>CPTU</td>
<td>After Embankment Construction</td>
</tr>
<tr>
<td>CPT 36</td>
<td>GTTH9516C -83</td>
<td>CPTU</td>
<td>After Embankment Construction</td>
</tr>
</tbody>
</table>
Max. Depth: 12.30 (m)

Depth Inc.: 0.05 (m)

SIT: Soil Behavior Type (Robertson and Campanella 1988)
Site: GTTH9516C-27
Location: J-RT
Crne: 2.5 TON A 002
Date: 05:1995 14:49

Depth (m)

Max. Depth: 11.95 (m)
Depth Inc.: 0.05 (m)

SBT: Soil Behavior Type (Robertson and Campanella 1988)
Max. Depth: 12.30 (m)
Depth Inc.: 0.05 (m)

SIT: Soil Behavior Type (Robertson and Campanella 1988)
Site: GTTH9516C-32
Location: J PIT
Date: 05/26/95 12:18

Max. Depth: 11.50 (m)
Depth Inc.: 0.05 (m)

SST: Soil Behavior Type (Robertson and Campanella 1980)
APPENDIX L

Phase III Shear Wave Velocity Results
(Iravani et al., 1995; after ConeTec, 1995)

N.B. An extensive SCPT investigation was carried out across the entire Phase III site. The $V_s$ results presented here are those from or close to the detailed site characterization area.
Conetec Investigations Ltd.

Client: CANLEX
Location: Syncrude, Ft. McMurray, J-Pit
Hole: GTTH9516C-31 (ConeTec #144CPT26)
Shear Source Offset .51m / Seismometer .20m above 1
Source: Hammer and Beam
Cone: AD002 / 2.5 tonne
May 26, 1995

<table>
<thead>
<tr>
<th>Geophone Depth (m)</th>
<th>Distance (m)</th>
<th>Last Interval Time (msec)</th>
<th>Average Depth of sensor (m)</th>
<th>Shear Wave Velocity (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.6</td>
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<td>84</td>
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<td>4.1</td>
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<td>9.4</td>
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<td>7.6</td>
<td>7.62</td>
<td>8.0</td>
<td>7.1</td>
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<tr>
<td>8.6</td>
<td>8.62</td>
<td>6.8</td>
<td>8.1</td>
<td>146</td>
</tr>
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<td>12.11</td>
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<td>11.85</td>
<td>82</td>
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</table>
ConeTec Investigations Ltd.

Client: CANLEX
Location: Syncrude, Ft. McMurray, J-Pit
Hole: GTTH9516C-32 (ConeTec # 144CPT27)
Shear Source Offset 1.6m, 2.4 m Below Surface
Seismometer .20m above tip
Source: Hammer and Auger
Cone: AD002 / 2.5 tonne
May 26, 1995

<table>
<thead>
<tr>
<th>Geophone Depth (m)</th>
<th>Distance (m)</th>
<th>Last Interval Time (msec)</th>
<th>Average Depth of sensor (m)</th>
<th>Shear Wave Velocity (m/sec)</th>
</tr>
</thead>
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<td>6.6</td>
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<td>9.2</td>
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<td>11.0</td>
<td>118</td>
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</table>
APPENDIX M

Phase III Geophysical Results
(Iravani et al., 1995; after Skirrow, 1995)
Calculating Void Ratio, $e$

Century Geophysical Tool 9036AA and 9071A
Data smoothed over 5 points (i.e. 80 mm)

FIGURE 10
CALCULATED VOID RATIO
Runs #1, #2, and #3
Century Geophysical Tool 9036AA and 9071A
Data smoothed over 5 points (i.e. 80 mm)

FIGURE 11
CALCULATED VOID RATIO
Runs #1, #2, and #3

AGRA
Earth & Environmental
FIGURE 12 COMPARISON OF AVERAGE CALCULATED VOID RATIOS

Frozen core sample void ratio data supplied by University of Alberta August 18, 1995

AGRA
Earth & Environmental
FIGURE 12b  COMPARISON OF AVERAGE CALCULATED VOID RATIOS
APPENDIX N

Phase III Pressuremeter Results
(Hughes, 1996)
<table>
<thead>
<tr>
<th>Test Location</th>
<th>Depth to Centre (m)</th>
<th>File Number</th>
</tr>
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<tbody>
<tr>
<td>SBPMT1</td>
<td>2.13</td>
<td>cant8.av</td>
</tr>
<tr>
<td></td>
<td>2.89</td>
<td>cant9.av</td>
</tr>
<tr>
<td></td>
<td>3.65</td>
<td>cant10.av</td>
</tr>
<tr>
<td></td>
<td>4.53</td>
<td>cant11.av</td>
</tr>
<tr>
<td></td>
<td>5.18</td>
<td>cant12.av</td>
</tr>
<tr>
<td></td>
<td>5.94</td>
<td>cant13.av</td>
</tr>
<tr>
<td></td>
<td>6.87</td>
<td>cant14.av</td>
</tr>
<tr>
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<td>7.46</td>
<td>cant15.av</td>
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<td></td>
<td>8.23</td>
<td>cant16.av</td>
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</table>

Table I. Self-bored pressuremeter tests at location SBPMT1

<table>
<thead>
<tr>
<th>Test Location</th>
<th>Depth to Centre (m)</th>
<th>File Number</th>
</tr>
</thead>
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<td>cant1.av</td>
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<tr>
<td></td>
<td>2.89</td>
<td>cant2.av</td>
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<tr>
<td></td>
<td>3.65</td>
<td>cant3.av</td>
</tr>
<tr>
<td></td>
<td>4.53</td>
<td>cant4.av</td>
</tr>
<tr>
<td></td>
<td>5.18</td>
<td>cant5.av*</td>
</tr>
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<td></td>
<td>5.94</td>
<td>cant6.av</td>
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<tr>
<td></td>
<td>6.87</td>
<td>cant7.av</td>
</tr>
</tbody>
</table>

* The pressure regulator was turned up too fast at the start of the test. As a result, the data acquisition system did not respond rapidly enough to capture the start of the test.

Table II. Self-bored pressuremeter tests at location SBPMT2
<table>
<thead>
<tr>
<th>Test Location</th>
<th>Depth to Centre (m)</th>
<th>File Number</th>
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</tr>
<tr>
<td></td>
<td>5.18</td>
<td>cant19.av</td>
</tr>
</tbody>
</table>

* Five large-cycle cyclic pressure-expansion tests

**Table III.** Self-bored pressuremeter tests at location SBPMT3

<table>
<thead>
<tr>
<th>Test Location</th>
<th>Depth to Centre (m)</th>
<th>Maximum Shear Modulus G (kPa)</th>
<th>Pressure at 10% radial strain (kPa)</th>
<th>File Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBPMT1</td>
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<td>1,100</td>
<td>33</td>
<td>cant8.av</td>
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<tr>
<td></td>
<td>2.89</td>
<td>7,000</td>
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<td>cant9.av</td>
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<tr>
<td></td>
<td>3.65</td>
<td>5,500</td>
<td>133</td>
<td>cant10.av</td>
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<td></td>
<td>4.53</td>
<td>7,300</td>
<td>185</td>
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<td>17,500</td>
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**Table IV.** Shear Modulus and Pressure at 10% strain at location SBPMT1
<table>
<thead>
<tr>
<th>Test Location</th>
<th>Depth to Centre (m)</th>
<th>Maximum Shear Modulus G (kPa)</th>
<th>Pressure at 10% radial strain (kPa)</th>
<th>File Number</th>
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</thead>
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**Table V.** Shear Modulus and Pressure at 10% strain at location SBPMT2

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<th>Test Location</th>
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<th>Maximum Shear Modulus G (kPa)</th>
<th>Pressure at 10% radial strain (kPa)</th>
<th>File Number</th>
</tr>
</thead>
<tbody>
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<td>3.64</td>
<td>-</td>
<td>140</td>
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</tr>
<tr>
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<td>4.53</td>
<td>-</td>
<td>175</td>
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<td>5.18</td>
<td>-</td>
<td>170?</td>
<td>cant19.av(1)</td>
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**Table VI.** Shear Modulus and Pressure at 10% strain at location SBPMT3
### Pressuremeter Data

**Syneride Trial Embankment**

<table>
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<th>Hole No.</th>
<th>5bpm1</th>
<th>Depth 2.05 m</th>
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</table>

**Canlex Stage III**

- 1 June 1995
- File: D:\DATA\CANT.O\CANT8.RP

### Graph

- **Graph Title:** Shear Modulus 1075 kPa

**Graph Description:**

- **Pressure (kPa):** 0, 25, 50, 75, 100
- **Radial Displacement/Radius (%):** 0, 3, 6, 9, 12

- **Lines:**
  - **Field Data**
  - **Shear Modulus**
### Pressuremeter Data

<table>
<thead>
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<th>Syncrude Trial Embankment</th>
<th>1 June 1995</th>
</tr>
</thead>
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<td>Hole No. SHPM1</td>
<td>Depth 2.89 m to centre</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Pressure (kPa)</th>
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<th>200</th>
<th>300</th>
<th>400</th>
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<tbody>
<tr>
<td>Radial Displacement/Radius(%)</td>
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<td>6</td>
<td>9</td>
<td>12</td>
</tr>
</tbody>
</table>

- **Field Data**
- **Shear Modulus**

Shear Modulus 7029 kPa
Pressuremeter Data

Synerude Trial Embankment
Hole No. SBPM1

Canlex Stage III
1 June 1995
Depth 3.64 m to centre

Field Data
Shear Modulus

Pressure (kPa)

Radial Displacement/Radius(%)
Pressuremeter Data

Canlex Stage III

Syncrude Trial Embankment
1 June 1995

Hole No.: SBPM1
Depth 4.53 m to centre
File D:\DATA\CAN\LO\CANT11.P

<table>
<thead>
<tr>
<th>Pressure (kPa)</th>
<th>Radial Displacement/Radius(%)</th>
</tr>
</thead>
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<td>100</td>
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<td>150</td>
<td>9</td>
</tr>
<tr>
<td>200</td>
<td>12</td>
</tr>
</tbody>
</table>

Field Data
Shear Modulus

Shear Modulus 7283 kPa

Hughes
Pressuremeter Data

Syncrude Trial Embankment
Hole No. SBPM1
Depth 5.18 m to centre

Canlex Stage III
1 June 1995
File D:\DATA\CANLO\CANT12.P

<table>
<thead>
<tr>
<th>Pressure (kPa)</th>
<th>Radial Displacement/Radius(%)</th>
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</thead>
<tbody>
<tr>
<td>Field Data</td>
<td>Shear Modulus</td>
</tr>
</tbody>
</table>

Shear Modulus 17487 kPa

Hughes
Pressuremeter Data

Canlex Stage III

Synerude Trial Embankment
1 June 1995

Hole No. SBPM1 Depth 5.94 m to centre

File D:\DATA\CANLO\CANT13.P

Pressure (kPa)

Field Data

Shear Modulus

Shear Modulus 16281 kPa

Radial Displacement/Radius(%)
Pressuremeter Data Canlex Stage III
Syncrude Trip Embankment 1 June 1995
Hole No. SBPM1 Depth 6.87 to centre File D:\DATA\CANLO\CANT14.P

---

- Field Data
- Shear Modulus

Shear Modulus 8761 kPa

HUGHES
### Pressuremeter Data

<table>
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<tr>
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</thead>
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<td>SBPM1</td>
<td>7.46m to centre</td>
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</table>

---

### Canlex Stage III

1 June 1995

---

#### Graph

- **Y-axis**: Pressure (kPa)
- **X-axis**: Radial Displacement/Radius (%)

- **Line 1**: Field Data
- **Line 2**: Shear Modulus

Shear Modulus 0 kPa

---

Hughes
Pressuremeter Data

Syncrude Trial Embankment
Hole No. SBPM1
Depth 8.23 m to centre

1 June 1995

File D:\DATA\CANT16.P

Pressure (kPa)

Field Data
Shear Modulus

Radial Displacement/Radius(%)
## Pressuremeter Data

**Syncrude Trial Embankment**

<table>
<thead>
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<th>SnPM2</th>
<th>Depth 2.13 m to centre</th>
<th>File D:DATA\CANLO\CANT1.P</th>
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<table>
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<th>Pressure (kPa)</th>
<th>Radial Displacement/Radius(%)</th>
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<tr>
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<td>0</td>
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<td>300</td>
<td>9</td>
</tr>
<tr>
<td>400</td>
<td>12</td>
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</table>

Shear Modulus: 6251 kPa

HUGHES
Pressuremeter Data

Syncrude Trial Embankment

Hole No. SBPM2
Depth 2.89 m to centre

Canlex Stage III

31 May 95

Field Data

Shear Modulus

Pressure (kPa)

300

200

100

0

Radial Displacement/Radius(%) 0 3 6 9 12

Shear Modulus 8571 kPa

Hughes
Pressuremeter Data

Synergude Trial Embankment

Hole No. SBPM2 Depth 3.65 m to centre

Canlex Stage III

31 May 95

File D:\DATA\CANLO\CANT3.P

--- Field Data
--- Shear Modulus

Shear Modulus 9191 kPa

Pressure (kPa)

Radial Displacement/Radius(%)
Pressuremeter Data

Canlex Stage III

Synerude Trial Embankment
Hole No. SBPM2 Depth 4.53 m to centre

File D:\DATA\CANLO\CANT4.P

Pressure (kPa)

Shear Modulus 7283 kPa

Hughes
Pressuremeter Data

Canex Stage III

Synerdne Trial Embankment
Hole No. SBPM2 Depth 5.18 m to centre

31 May 31

File D:\DATA\CANLO\CANT5.P

--- Field Data
--- Shear Modulus

Pressure (kPa)

Shear Modulus 9866 kPa

Radial Displacement/Radius(%)
Pressuremeter Data

<table>
<thead>
<tr>
<th>Syncrude Trial Embankment</th>
<th>Canlex Stage III</th>
</tr>
</thead>
<tbody>
<tr>
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<td>Depth 5.94 m to centre</td>
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<table>
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<th>Radial Displacement/Radius(%)</th>
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<tbody>
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<td>100</td>
<td>3</td>
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<td>300</td>
<td>9</td>
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<tr>
<td>400</td>
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Field Data
Shear Modulus

Shear Modulus 14884 kPa
### Pressuremeter Data

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<th>150</th>
<th>300</th>
<th>450</th>
<th>600</th>
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<td>Radial Displacement/Radius(%)</td>
<td>0</td>
<td>3</td>
<td>6</td>
<td>9</td>
<td>12</td>
</tr>
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</table>

#### Syncrude Trial Embankment
- **Hole No.:** SBPM2
- **Depth:** 6.87 m to centre

### Canlex Stage III
- **Date:** 31 May 95
- **File:** D:\DATA\CANLO\CANT7.P

### Graph
- **Shear Modulus:** 27575 kPa
Pressuremeter Data
Syncrude Trial Embankment
1 June 1995
Hole No. SBPM3 Depth 3.65 m to centre
File D:\DATA\CANLO\CANT7X.P

<table>
<thead>
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<th>Pressure (kPa)</th>
<th>Radial Displacement/Radius(%)</th>
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<td>0</td>
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<td>150</td>
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</tr>
<tr>
<td>200</td>
<td>12</td>
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</table>

Field Data
Shear Modulus

Shear Modulus 0 kPa

Hughes
Pressuremeter Data  
Canlex Stage III  

Synerude Trial Embankment  
1 June 1995  

Hole No. Shpm3  
Depth 5.18 m to centre  

Field Data  
Shear Modulus  

Pressure (kPa)  

Radial Displacement/Radius(%)  

Shear Modulus 0 kPa  

Hughes
Figure 3. Range of Pressuremeter test data from tests Cant4, Cant11 and Cant18, all at 4.53 m
Figure 4. Ideal Pressure Expansion Curve at the 4.5 m level for constant volume deformation

(Assuming a critical state friction angle of 30 degrees and an initial secant shear modulus of 3,600 kPa for a water level at the surface and at 1m below)
APPENDIX O

Phase III U.B.C. Laboratory Testing Results
(Vaid et al., 1996)
NOTES:

- $G_s = 2.66$
- Some specimens collapsed during saturation process following thawing
- Specimen FS52C6B3 suffered some collapse during saturation.
- Possible end slip in monotonic simple shear test SP3-13.
a) Undisturbed Phase III samples tested in triaxial compression and extension, simple shear and cyclic simple shear.
# CANLEX - Phase III

## Triaxial Tests on Frozen Syncrude Sand

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sample No</th>
<th>$e_i$</th>
<th>Thaw</th>
<th>B (%)</th>
<th>Depth (mm)</th>
<th>$\sigma'_{vc}$ (kPa)</th>
<th>$\sigma'_{hc}$ (kPa)</th>
<th>Consolidation</th>
<th>$\Delta h$ (mm)</th>
<th>$e_c$</th>
<th>Static Test</th>
<th>Test Type</th>
<th>Contractive Dilative</th>
<th>$S_{P200}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tr>
<tr>
<td>4.52</td>
<td>FS5 C4A1</td>
<td>0.711</td>
<td>-0.21</td>
<td>100</td>
<td>0.715</td>
<td>48</td>
<td>24</td>
<td>0.223</td>
<td>0.714</td>
<td>TC</td>
<td>D</td>
<td></td>
<td></td>
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<tr>
<td>4.65</td>
<td>FS5 C4A2</td>
<td>0.726</td>
<td>-0.13</td>
<td>100</td>
<td>0.727</td>
<td>48</td>
<td>24</td>
<td>0.212</td>
<td>0.725</td>
<td>TC</td>
<td>D</td>
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<tr>
<td>6.66</td>
<td>FS5 C6B3</td>
<td>0.787</td>
<td>-0.12</td>
<td>100</td>
<td>0.775</td>
<td>66</td>
<td>33</td>
<td>0.307</td>
<td>0.772</td>
<td>TC</td>
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<tr>
<td>3.76</td>
<td>FS4 C1 3(A)</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.76</td>
<td>FS4 C1 3(B)</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
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<tr>
<td>4.48</td>
<td>FS26 C3 I(A)</td>
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<td></td>
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<tr>
<td>4.48</td>
<td>FS26 C3 I(B)</td>
<td>0.882</td>
<td>-0.07</td>
<td>99</td>
<td>0.874</td>
<td>112</td>
<td>56</td>
<td>0.263</td>
<td>0.868</td>
<td>TE</td>
<td>C</td>
<td>2</td>
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<tr>
<td>3.51</td>
<td>FS5 C1B 2A</td>
<td>0.877</td>
<td>0.71</td>
<td>99</td>
<td>0.839</td>
<td>80</td>
<td>40</td>
<td>0.235</td>
<td>0.827</td>
<td>TE</td>
<td>D</td>
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<tr>
<td>3.51</td>
<td>FS5 C1B 2B</td>
<td>0.881</td>
<td>0.48</td>
<td>97</td>
<td>0.852</td>
<td>42</td>
<td>21</td>
<td>0.872</td>
<td>0.847</td>
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<tr>
<td>3.51</td>
<td>FS5 C1B 2C</td>
<td>0.845</td>
<td>0.40</td>
<td>91</td>
<td>0.837</td>
<td>42</td>
<td>21</td>
<td>0.653</td>
<td>0.833</td>
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<td>3.66</td>
<td>FS5 C1B3 1</td>
<td>0.833</td>
<td>0.23</td>
<td>100</td>
<td>0.813</td>
<td>44</td>
<td>22</td>
<td>0.170</td>
<td>0.811</td>
<td>TE</td>
<td>C</td>
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<tr>
<td>3.66</td>
<td>FS5 C1B3 2</td>
<td>0.827</td>
<td>0.05</td>
<td>92</td>
<td>0.806</td>
<td>44</td>
<td>22</td>
<td>0.239</td>
<td>0.804</td>
<td>TC</td>
<td>D</td>
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</table>

- Cracks in sample.

Samples collapsed while applying differential vacuum.

File: C:\CANLEX\FINAL\T-F-P3.WP6

Created On: Apr 20, 1996.
**CANLEX - PHASE III**

**Triaxial Tests on Frozen Syncrude Sand**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sample No</th>
<th>(e_i)</th>
<th>(\Delta h) (mm)</th>
<th>B (%)</th>
<th>(e_{20}) kPa</th>
<th>(\sigma'_{vc}) (kPa)</th>
<th>(\sigma'_{hc}) (kPa)</th>
<th>Consolidation (\Delta h) (mm)</th>
<th>(e_c)</th>
<th>Static Test</th>
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</thead>
<tbody>
<tr>
<td>3.66</td>
<td>FS5 C1B3 3</td>
<td>0.824</td>
<td>0.08</td>
<td>96</td>
<td>0.822</td>
<td>204</td>
<td>102</td>
<td>0.846</td>
<td>0.811</td>
<td>TC C 61</td>
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<td>0.822</td>
<td>0.22</td>
<td>92</td>
<td>0.809</td>
<td>204</td>
<td>102</td>
<td>0.972</td>
<td>0.796</td>
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<td>2.95</td>
<td>FS6 C2B2 1</td>
<td>0.673</td>
<td>0.01</td>
<td>99</td>
<td>0.668</td>
<td>294</td>
<td>147</td>
<td>0.878</td>
<td>0.657</td>
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<td>FS6 C2B2 2</td>
<td>0.698</td>
<td>0.05</td>
<td>98</td>
<td>0.692</td>
<td>194</td>
<td>97</td>
<td>0.710</td>
<td>0.682</td>
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<td>FS6 C2B2 3</td>
<td>0.727</td>
<td>0.10</td>
<td>NA</td>
<td>0.723</td>
<td>48</td>
<td>24</td>
<td>0.420</td>
<td>0.722</td>
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<td>FS6 C2B2 4</td>
<td>0.696</td>
<td>0.07</td>
<td>NA</td>
<td>0.680</td>
<td>48</td>
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<td>0.149</td>
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<td>98</td>
<td>0.712</td>
<td>196</td>
<td>98</td>
<td>0.655</td>
<td>0.703</td>
<td>TC D</td>
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<td>3.23</td>
<td>FS26 C21 4</td>
<td>0.700</td>
<td>0.10</td>
<td>97</td>
<td>0.703</td>
<td>196</td>
<td>98</td>
<td>0.529</td>
<td>0.698</td>
<td>TE D</td>
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<td>FS26 C22 3</td>
<td>0.764</td>
<td>0.19</td>
<td>100</td>
<td>0.758</td>
<td>196</td>
<td>98</td>
<td>0.905</td>
<td>0.749</td>
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<td>FS26 C54 3</td>
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<td>68</td>
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<td>0.745</td>
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<td>99</td>
<td>0.739</td>
<td>100</td>
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<td>FS26 C42 3</td>
<td>0.746</td>
<td>-0.02</td>
<td>100</td>
<td>0.745</td>
<td>100</td>
<td>50</td>
<td>0.275</td>
<td>0.741</td>
<td>TE/C 2.5</td>
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</table>
Triaxial Test

Sample: FS52 C4A1

\( \sigma'_v = 48 \text{ kPa} \)
\( \sigma'_h = 24 \text{ kPa} \)
\( e_c = 0.714 \)

Monotonic Loading

Test No: TP31

\( \sigma'_d, \text{ kPa} \)

\( \Delta u/\sigma'_h \)

Cavitation

\( (\sigma'_1 - \sigma'_3)/2, \text{ kPa} \)

\( (\sigma'_1 + \sigma'_3)/2, \text{ kPa} \)
Triaxial Test
Sample: FS52 C4A2

Monotonic Loading
Test No: TP32

$\sigma'_{vc} = 48$ kPa
$\sigma'_{hc} = 24$ kPa
$\varepsilon_c = 0.725$

Cavitation

$(\sigma'_1 - \sigma'_3)/2$, kPa

$(\sigma'_1 + \sigma'_3)/2$, kPa
Triaxial Test
Sample: FS52 C683

\[ \sigma'_{vc} = 66 \text{ kPa} \]
\[ \sigma'_{hc} = 33 \text{ kPa} \]
\[ e_c = 0.772 \]

Monotonic Loading
Test No: TP33

\[ (\sigma'_1 + \sigma'_3)/2 \text{, kPa} \]
\[ (\sigma'_1 - \sigma'_3)/2 \text{, kPa} \]
Triaxial Test
Sample: FS26 C31(B)

\[ \sigma'_c = 112 \text{ kPa} \]
\[ \sigma'_h = 56 \text{ kPa} \]
\[ e_c = 0.868 \]

Monotonic Loading
Test No: TP37

\( \sigma_d, \text{kPa} \)
\( \Delta u/\sigma'_h \)
\( \varepsilon_a \)
\( \frac{(\sigma'_1 - \sigma'_3)}{2}, \text{kPa} \)

\( (\sigma'_1 + \sigma'_3)/2, \text{kPa} \)
Triaxial Test

Sample: FS5 C1B 2A

Monotonic Loading

Test No: TP38

\[ \sigma_{vc} = 80 \text{ kPa} \]
\[ \sigma'_{hc} = 40 \text{ kPa} \]
\[ e_c = 0.827 \]

\( \sigma_d, \text{ kPa} \)

\( \Delta u/\sigma'_{hc} \)

\( \varepsilon_a \)

\( (\sigma'_1 - \sigma'_3)/2, \text{ kPa} \)

\( (\sigma'_1 + \sigma'_3)/2, \text{ kPa} \)

Necking
Monotonic Loading
Test No: TP39

Triaxial Test
Sample : FS5 C1B 2B

\( \sigma_{vc} = 42 \text{ kPa} \)
\( \sigma_{hc} = 21 \text{ kPa} \)
\( e_c = 0.847 \)
Triaxial Test
Sample: FS5 C1B 2C

\[ \sigma'_{vc} = 42 \text{ kPa} \]
\[ \sigma'_{hc} = 21 \text{ kPa} \]
\[ e_c = 0.833 \]

Necking

Monotonic Loading
Test No: TP3A

\[ \Delta \sigma / \sigma'_{hc} \]

\[ (\sigma'_{1} - \sigma'_{3})/2, \text{kPa} \]

\[ (\sigma'_{1} + \sigma'_{3})/2, \text{kPa} \]
Triaxial Test
Sample: FS5 C1B3 1
\[ \sigma''_c = 44 \text{ kPa} \]
\[ \sigma''_h = 22 \text{ kPa} \]
\[ e_c = 0.811 \]

Monotonic Loading
Test No: TP3B

\( \sigma_d \), kPa

\( \Delta u / n_h \), kPa

\( \epsilon_a \)

\( (\sigma'_1 - \sigma'_3)/2 \), kPa

\( (\sigma'_1 + \sigma'_3)/2 \), kPa
Monotonic Loading

Test No: TP3C

Triaxial Test

Sample: FS5 C1B3 2

\( \sigma'_{\text{vc}} = 44 \text{ kPa} \)

\( \sigma'_{\text{hc}} = 22 \text{ kPa} \)

\( e_c = 0.804 \)

\[ \begin{align*}
\sigma_d, \text{ kPa} & \\
\Delta u/\sigma'_{\text{hc}} & \\
(\sigma'_{1} - \sigma'_{3})/2, \text{ kPa} & \\
(\sigma'_{1} + \sigma'_{3})/2, \text{ kPa} & 
\end{align*} \]
Monotonic Loading
Test No: TP3D

Triaxial Test
Sample: FS5 C1B3 3

\[ \sigma_{vc} = 204 \text{ kPa} \]
\[ \sigma_{hc} = 102 \text{ kPa} \]
\[ e_c = 0.811 \]
Monotonic Loading
Test No: TP3E

Triaxial Test
Sample: FS5 C1B3 4

$\sigma'_{vc} = 204$ kPa
$\sigma'_{hc} = 102$ kPa
$e_c = 0.796$

Necking

$(\sigma'_1 - \sigma'_3)/2$, kPa

$(\sigma'_1 + \sigma'_3)/2$, kPa
Triaxial Test
Sample: FS6 C2B2 1

Monotonic Loading
Test No: TP3F

\[ \sigma'_{vc} = 294 \text{ kPa} \]
\[ \sigma'_{hc} = 147 \text{ kPa} \]
\[ e_c = 0.657 \]

\[ \sigma_d, \text{ kPa} \]

\[ \Delta u/\sigma'_{hc} \]

\[ \varepsilon_a \]

\[ (\sigma'_1 - \sigma'_3)/2, \text{ kPa} \]

\[ (\sigma'_1 + \sigma'_3)/2, \text{kPa} \]
Triaxial Test
Sample: FS6 C2B2 2

\[ \sigma'_{vc} = 194 \text{ kPa} \]
\[ \sigma'_{hc} = 97 \text{ kPa} \]
\[ e_c = 0.682 \]

Load Cell Saturated.

Monotonic Loading
Test No: TP3G

\[ \frac{\Delta \bar{u}}{\sigma'_c} \]

\[ \varepsilon_a \]

\[ \frac{(\sigma'_1 - \sigma'_3)}{2}, \text{kPa} \]

\[ (\sigma'_1 + \sigma'_3)/2, \text{kPa} \]}
Triaxial Test
Sample: FS6 C2B2 3

Monotonic Loading
Test No: TP3H

\( \sigma'_{vc} = 48 \text{ kPa} \)
\( \sigma'_{hc} = 24 \text{ kPa} \)
\( e_c = 0.722 \)

Necking
Triaxial Test

Sample: FS6 C2B2 4

Monotonic Loading

Test No: TP31

$\sigma_{vc} = 48\, \text{kPa}$

$\sigma_{hc} = 24\, \text{kPa}$

$\varepsilon_c = 0.680$

$\Delta \varepsilon / \sigma_{hc}$

Cavitation

$\varepsilon_a$

$(\sigma'_1 - \sigma'_3)/2, \text{kPa}$

$(\sigma'_1 + \sigma'_3)/2, \text{kPa}$
Triaxial Test
Sample: FS26 C21 1
Test No: TP3J

\[ \sigma_{vc} = 40 \text{ kPa} \]
\[ \sigma_{hc} = 20 \text{ kPa} \]
\[ e_c = 0.708 \]

Necking

Cavitation

\[ (\sigma_1 - \sigma_3)/2, \text{kPa} \]

\[ (\sigma_1 + \sigma_3)/2, \text{kPa} \]
Monotonic Loading
Test No: TP3K

Triaxial Test
Sample: FS26 C21 2

\[ \sigma'_{vc} = 40 \text{ kPa} \]
\[ \sigma'_{hc} = 20 \text{ kPa} \]
\[ e_c = 0.691 \]
Triaxial Test
Sample: FS26 C21 3
Test No: TP3L

Monotonic Loading

\[ \sigma_{vc} = 196 \text{ kPa} \]
\[ \sigma_{hc} = 98 \text{ kPa} \]
\[ e_c = 0.703 \]

Load Cell Saturated

\[
\begin{align*}
\sigma_d, \text{ kPa} & \quad \Delta \mu/\sigma', \text{ kPa} \\
0 & \quad 0.0
\end{align*}
\]

\[
\varepsilon_a
\]

\[
\left(\sigma'_1 - \sigma'_3\right)/2, \text{ kPa}
\]

\[
\left(\sigma'_1 + \sigma'_3\right)/2, \text{ kPa}
\]
Monotonic Loading

Test No: TP3M

Sample: FS26 C21 4

- $\sigma'_v = 196$ kPa
- $\sigma'_h = 98$ kPa
- $e_e = 0.698$

Triaxial Test

Necking
Triaxial Test
Sample: FS26 C22 3

Monotonic Loading
Test No: TP3N

$\sigma'_{vc} = 196$ kPa
$\sigma'_{hc} = 98$ kPa
$e_c = 0.749$

Graphs showing stress-strain relationship in triaxial test.
Triaxial Test
Sample: FS26 C54 3
Test No: TP30

\[ \sigma'_c = 68 \text{ kPa} \]
\[ \sigma'_h = 34 \text{ kPa} \]
\[ e_c = 0.745 \]
Triaxial Test
Sample: FS26 C42 2

Monotonic Loading
Test No: TP3P

$\sigma'_v = 100$ kPa
$\sigma'_h = 50$ kPa
$e_c = 0.734$
Triaxial Test

Sample: FS26 C423

Test No: TP3Q

\[ \sigma'_{vc} = 100 \text{ kPa} \]

\[ \sigma'_{hc} = 50 \text{ kPa} \]

\[ e_c = 0.741 \]
### Monotonic Simple Shear Tests on Frozen Syncrude Sand

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sample No</th>
<th>( \epsilon_{\text{i}} )</th>
<th>( \epsilon_{\text{21kPa}} )</th>
<th>( \sigma'_{\text{vc}} ) (kPa)</th>
<th>( \epsilon_{\text{static}} ) (kPa)</th>
<th>( \epsilon_{\text{c}} )</th>
<th>Static Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.39</td>
<td>FS26 C22 1A</td>
<td>0.776</td>
<td>0.751</td>
<td>100</td>
<td>0</td>
<td>0.737</td>
<td>C</td>
</tr>
<tr>
<td>3.41</td>
<td>FS26 C22 1B</td>
<td>0.790</td>
<td>0.751</td>
<td>100</td>
<td>0</td>
<td>0.735</td>
<td>D</td>
</tr>
<tr>
<td>3.44</td>
<td>FS26 C22 1C</td>
<td>0.791</td>
<td>0.749</td>
<td>100</td>
<td>0</td>
<td>0.733</td>
<td>D</td>
</tr>
<tr>
<td>3.47</td>
<td>FS26 C22 1D</td>
<td>0.797</td>
<td>0.751</td>
<td>100</td>
<td>0</td>
<td>0.738</td>
<td>D</td>
</tr>
<tr>
<td>3.39</td>
<td>FS26 C22 2A</td>
<td>0.766</td>
<td>0.729</td>
<td>100</td>
<td>0</td>
<td>0.716</td>
<td>D</td>
</tr>
<tr>
<td>3.41</td>
<td>FS26 C22 2B</td>
<td>0.779</td>
<td>0.741</td>
<td>100</td>
<td>0</td>
<td>0.727</td>
<td>C</td>
</tr>
<tr>
<td>3.45</td>
<td>FS26 C22 2C</td>
<td>0.777</td>
<td>0.728</td>
<td>100</td>
<td>0</td>
<td>0.714</td>
<td>D</td>
</tr>
<tr>
<td>3.47</td>
<td>FS26 C22 2D</td>
<td>0.783</td>
<td>0.736</td>
<td>100</td>
<td>0</td>
<td>0.721</td>
<td>D</td>
</tr>
</tbody>
</table>

\( \epsilon_{\text{i}} \) : Calculated from final dry weight of solids and UoA dimensions

\( \epsilon_{\text{21kPa}}, \epsilon_{\text{c}} \) : Calculated from final dry weight of solids and UBC dimensions

File: C:\CANLEX\PHASE.3\SS\MON-P3.WP6

Created On: Apr 20, 1996.
Simple Shear Test

Monotonic Loading

Sample: FS26 C2 2 1A

Test No: SP3-01

\( \sigma'_{vc} = 100 \text{ kPa} \)

\( e_c = 0.737 \)

\( \gamma \), %

\( \tau \), kPa

\( \Delta u/\sigma'_{vo} \)
Simple Shear Test

Sample: FS26 C22 1B

Monotonic Loading

Test No: SP3-02

\( \sigma'_v = 100 \text{ kPa} \)

\( e_c = 0.735 \)

\( \Delta u / \sigma'_v \)

\( \gamma, \% \)

\( \tau, \text{kPa} \)

\( \sigma'_v, \text{kPa} \)
Simple Shear Test

Test No: SP3-03

Sample: FS26 C2 21C

\( \sigma'_{vc} = 100 \text{ kPa} \)

\( e_c = 0.733 \)

Monotonic Loading

\( \tau, \text{kPa} \)

\( \Delta u / \sigma'_{vo} \)

\( \gamma, \% \)

\( \sigma'_{vc}, \text{kPa} \)

\( \tau, \text{kPa} \)
Simple Shear Test

Sample: FS26 C2 2 1D
Test No: SP3-04

\( \sigma'_{vc} = 100 \text{ kPa} \)
\( e_c = 0.738 \)

\[ \Delta u / u'_{vo} \]

\[ \tau, \text{ kPa} \]

\[ \gamma, \% \]

\[ \sigma', \text{ kPa} \]
Simple Shear Test

Monotonic Loading

Sample: FS26 C22 2A
Test No: SP3-05

\( \sigma'_{vc} = 100 \text{ kPa} \)

\( e_c = 0.716 \)
Simple Shear Test

Monotonic Loading

Sample: FS26 C22 2B
Test No: SP3-06

\( \sigma'_{vc} = 100 \, \text{kPa} \)

\( e_c = 0.727 \)
Simple Shear Test

Sample: FS26 C22 2C

Monotonic Loading

Test No: SP3-07

\[ \sigma''_{vc} = 100 \text{ kPa} \]

\[ e_c = 0.714 \]

\[ \Delta u / \sigma''_{vo} \]

\[ \tau, \text{ kPa} \]

\[ \gamma, \% \]

\[ \tau, \text{ kPa} \]

\[ \sigma''_{vc}, \text{ kPa} \]
Simple Shear Test

Monotonic Loading

Sample: FS26 C22 2D
Test No: SP3-13

\( \sigma'_v = 100 \text{ kPa} \)
\( e_c = 0.721 \)

\( \tau, \text{ kPa} \)

\( \Delta \omega/\sigma'_v \)

\( \gamma, \% \)

\( \tau, \text{ kPa} \)

\( \sigma'_v, \text{ kPa} \)
## Canlex - Phase III

### Cyclic Simple Shear Tests on Frozen Syncrude Sand

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sample No</th>
<th>$e_i$</th>
<th>$e_{20kPa}$</th>
<th>$\sigma_{ve}^*$ (kPa)</th>
<th>$\tau_{static}$ (kPa)</th>
<th>$e_r$</th>
<th>$\tau_y/\sigma_{ve}^*$</th>
<th>N</th>
<th>$\gamma_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.56</td>
<td>FS26 C42 1A</td>
<td>0.772</td>
<td>0.741</td>
<td>217</td>
<td>0</td>
<td>0.716</td>
<td>0.116</td>
<td>4</td>
<td>12.0</td>
</tr>
<tr>
<td>5.58</td>
<td>FS26 C42 1B</td>
<td>0.780</td>
<td>0.748</td>
<td>217</td>
<td>0</td>
<td>0.725</td>
<td>0.101</td>
<td>8</td>
<td>11.8</td>
</tr>
<tr>
<td>5.64</td>
<td>FS26 C42 1C</td>
<td>0.812</td>
<td>0.769</td>
<td>217</td>
<td>0</td>
<td>0.741</td>
<td>0.082</td>
<td>12</td>
<td>8.6</td>
</tr>
<tr>
<td>5.66</td>
<td>FS26 C42 1D</td>
<td>0.801</td>
<td>0.752</td>
<td>217</td>
<td>0</td>
<td>0.729</td>
<td>0.066</td>
<td>87</td>
<td>6.0</td>
</tr>
<tr>
<td>6.78</td>
<td>FS26 C54 1A</td>
<td>0.792</td>
<td>0.736</td>
<td>217</td>
<td>0</td>
<td>0.712</td>
<td>0.080</td>
<td>13</td>
<td>12.8</td>
</tr>
</tbody>
</table>

[File: C:\CANLEX\PHASE.3\SS\CYC-P3.WP6] Created On: April 20, 1996.
Simple Shear Test

Sample ID: FS26 C42 1A

Depth = 5.57 m

$\sigma'_{vc} = 217$ kPa

$e_c = 0.716$

$\tau_{cy}/\sigma'_{vc} = 0.116$

Graphs showing:

- $\tau_{cy},$ kPa
- $\Delta u/\sigma'_{vc}$
- $\gamma$, %

No of Cycles

SP3-08
Simple Shear Test

Sample ID: FS26 C42 1B
Depth = 5.59 m

\( \sigma'_{vc} = 217 \text{ kPa} \)
\( e_c = 0.725 \)

\( \tau_{cy} / \sigma'_{vc} = 0.101 \)

\( \Delta u / \sigma'_{vc} \)

\( \gamma \)
Simple Shear Test

Sample ID: FS26 C42 1C

Depth = 5.65 m

\[ \sigma'_{vc} = 217 \, \text{kPa} \]

\[ e_c = 0.741 \]

\[ \tau_{cy} / \sigma'_{vc} = 0.082 \]

No of Cycles

\[ \Delta \psi / \sigma'_{vc} \]

\[ \gamma \, \% \]
Simple Shear Test

Sample 1: FS26 C42 1D

Cyclic Loading

Depth = 5.67 m

\( \gamma, \% \)

\( \Delta u / \sigma'_{vc} \)

\( \tau_{cy}, \text{kPa} \)

- Depth = 5.67 m
- \( \sigma'_{vc} = 217 \text{kPa} \)
- \( e_c = 0.729 \)
- \( \gamma_c = 0.066 \)

No of Cycles

0 30

0 10

-30 -15 0 15 30

-30 -15 0 15 30

Sample ID: FS26 C42 1D
Simple Shear Test  Cyclic Loading

Sample ID: FS26 C54 1A  Depth = 6.78 m

$\sigma'_v = 217$ kPa
$\epsilon_c = 0.712$

$\tau_{cy} / \sigma'_v = 0.080$

$\Delta u / \sigma'_v$

No of Cycles

$\gamma$, %

No of Cycles

-357
Cyclic Simple Shear Tests.

Phase 3: Samples FS26 C42 1A to 1D & FS26 C54 1A

![Graph showing cyclic stress ratio vs. number of cycles for samples FS26 C42 1A to 1D & FS26 C54 1A.]

Key points:
- \( e_c = 0.716 \)
- \( e_c = 0.725 \)
- \( e_c = 0.741 \) (with a symbol indicating a specific measurement or condition)
- \( e_c = 0.729 \)
b) Reconstituted Phase III samples tested in simple shear and hollow cylinder torsion.
Monotonic Simple Shear Tests on Reconstituted Syncrude Sand

(Effect of Boiling the Sand prior to Reconstitution)

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test ID</th>
<th>$e_i$</th>
<th>$e_{20%}$</th>
<th>$\sigma'_{v,0}$ (kPa)</th>
<th>$\tau_c/\sigma'_{v,0}$</th>
<th>$e_c$</th>
<th>Static Test</th>
<th>$S_{FRS}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reconstituted as received</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>BS01</td>
<td>0.908</td>
<td>0.803</td>
<td>100</td>
<td>0.0</td>
<td>0.737</td>
<td>C</td>
<td>7.5</td>
</tr>
<tr>
<td>2</td>
<td>BS02</td>
<td>0.892</td>
<td>0.808</td>
<td>100</td>
<td>0.0</td>
<td>0.731</td>
<td>C</td>
<td>6.5</td>
</tr>
<tr>
<td>Reconstituted after Boiling</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>BS03</td>
<td>0.852</td>
<td>0.770</td>
<td>100</td>
<td>0.0</td>
<td>0.735</td>
<td>C</td>
<td>15.2</td>
</tr>
<tr>
<td>4</td>
<td>BS04</td>
<td>0.866</td>
<td>0.777</td>
<td>100</td>
<td>0.0</td>
<td>0.747</td>
<td>C</td>
<td>14.8</td>
</tr>
</tbody>
</table>

e_i, e_{20\%}, e_c: Calculated from final dry weight of solids and dimensions

Note: Two Tests were carried out to ensure the repeatability of tests.
Simple Shear Test

Test No: 01

\[ \sigma_{vo} = 100 \text{ kPa} \]
\[ e_c = 0.737 \]
Simple Shear Test

Test No: 02

\[ \sigma'_{ve} = 100 \text{ kPa} \]
\[ e_c = 0.731 \]
Simple Shear Test

Test No: 03

Boiled Sand

\[ \sigma_c' = 100 \text{ kPa} \]
\[ e_c = 0.735 \]

\[ \frac{\Delta l}{\sigma_v'} \]

\[ \gamma, \% \]

\[ \tau, \text{ kPa} \]

\[ \sigma_v', \text{ kPa} \]

363
Simple Shear Test

Test No: 04

Boiled Sand

\[ \sigma' = 100 \text{ kPa} \]
\[ \varepsilon_s = 0.747 \]
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Stress state at the end of consolidation</th>
<th>Undrained stress path</th>
<th>Stress state at peak deviator stress</th>
<th>Stress state at phase transformation or steady state</th>
</tr>
</thead>
<tbody>
<tr>
<td>SYN104</td>
<td>50 100 0.805</td>
<td>Triaxial compression</td>
<td>28.3 0.10</td>
<td>15.7 7.28</td>
</tr>
<tr>
<td>SYN113</td>
<td>50 100 0.808</td>
<td>Triaxial extension</td>
<td>7.8 0.32</td>
<td>8.2 9.60</td>
</tr>
<tr>
<td>SYN109</td>
<td>50 100 0.813</td>
<td>Plane strain compression  ( b = 0.4 )</td>
<td>30.0 0.11</td>
<td>16.7 5.40</td>
</tr>
<tr>
<td>SYN114</td>
<td>50 100 0.810</td>
<td>Plane strain extension  ( b = 0.4 )</td>
<td>9.8 0.40</td>
<td>7.5 8.82</td>
</tr>
<tr>
<td>SYN107</td>
<td>50 100 0.809</td>
<td>Torsional simple shear</td>
<td>---</td>
<td>6.7 8.68</td>
</tr>
</tbody>
</table>

\[
\sigma_d = \sigma_1 - \sigma_3
\]

\[
\cdot \ b = (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)
\]

\[
\cdot \cdot \ \epsilon_z = \epsilon_v = \epsilon_r = 0
\]
T.C. - Triaxial compression
T.E. - Triaxial extension
P.S.C. - Plane strain compression
P.S.E. - Plane strain extension
T.S.S. - Torsional simple shear
Repeatability of torsional simple shear test

\[
\sigma_d \text{ (kPa)}
\]

\[
\varepsilon_1 - \varepsilon_3 \text{ (\%)}
\]

\[
\sigma_{d/2} \text{ (kPa)}
\]

\[
\frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \text{ (kPa)}
\]
Repeatability of triaxial extension test

\[ \sigma_d \quad \text{(kPa)} \]

\[ \varepsilon_1 - \varepsilon_3 \quad \text{(\%)} \]

\[ \frac{\sigma_d}{2} \quad \text{(kPa)} \]

\[ \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \quad \text{(kPa)} \]
Repeatability of plane strain extension test

\[ \sigma_d \text{ (kPa)} \]

\[ \varepsilon_1 - \varepsilon_3 \text{ (\%)} \]

\[ \sigma_d/2 \text{ (kPa)} \]

\[ (\sigma'_1 + \sigma'_2 + \sigma'_3)/3 \text{ (kPa)} \]
APPENDIX P

Computer Aided Modelling (CAM)
A copy of a paper entitled “Strain level and uncertainty of liquefaction related index tests” (Roy et al., 1996). N.B. There was a small mistake in the original paper by Roy et al. (1996). The Massey undisturbed sample test results presented in the original paper were said to be for sample M94 F4 C4-2; however, they were actually for sample M94 F6 C7-A. This has been corrected in the version of the paper included here.
Strain Level and Uncertainty of Liquefaction Related Index Tests

D. Roy¹, R.G. Campanella², P.M. Byrne² and J.M.O. Hughes³

Abstract

From observations of the performance of natural deposits and man made earth structures under static and earthquake undrained loading conditions, liquefaction resistance has been correlated to large strain measurements, e.g., corrected Standard Penetration Test blow count, \( (N_t)_w \), and cone tip resistance, \( q_T \). The shear wave velocity can be correlated to \( (N_t)_w \) and \( q_T \) in some soils via weak statistical relationships. Therefore, shear wave velocity has also been proposed as an index of liquefaction resistance. Examination of data from three sites, however, shows that statistically tenable correlations do not exist between \( q_T \) and small strain shear modulus, \( G_{\text{max}} \). Consequently, shear wave velocity, a very low strain measurement closely related to \( G_{\text{max}} \), may not be an appropriate index of liquefaction resistance, which is a large strain phenomenon. In contrast, the statistical behavior of the pressure expansion curves measured in a self-boring pressuremeter test is quite similar to that of cone tip resistance at these sites. Thus, the self-boring pressuremeter data may be a viable alternative for assessment of liquefaction potential. An analytical procedure to derive the properties related to the undrained loading response of cohesionless soils from a self-boring pressuremeter test has been proposed. The procedure is illustrated with an example.

Introduction

Soil resistance to liquefaction can be directly estimated from laboratory testing of undisturbed sand samples obtained via ground freezing. However,

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³Hughes In-Situ Engineering, Inc., #804-938 Howe St., Vancouver, V6Z 1N9.
ground freezing is expensive to carry out and the sampling procedures may cause disturbance if executed without extreme care. As a result, indirect methods based on index tests are often employed for evaluation of liquefaction potential instead of laboratory tests. For instance, SPT (Standard Penetration Test) and CPTU (Piezocone Penetration Test) are used routinely as index tests for static and cyclic liquefaction resistance. The shear wave velocity (V_s) has also been suggested as an index of soil liquefaction resistance in static (Fear and Robertson, 1995) and cyclic (Robertson et al., 1991; Bierschwale and Stokoe, 1984) loading. An alternative approach has been proposed by Byrne et al. (1995b) which derives the volumetric behavior of the soil from inverse modeling of in-situ cavity expansion test. From the volumetric behavior, static liquefaction potential can be assessed for the undrained state. This paper examines the feasibility of the procedures based on shear wave velocity and in-situ cavity expansion for estimating liquefaction resistance.

Indirect Methods of Assessment of Liquefaction Potential

Examination of several case histories shows that the SPT blow count duly corrected for overburden pressure and driving energy, (N_1)_oo, can be related to the liquefaction resistance of soils (Seed, 1979). This approach has been accepted as a standard procedure for evaluation of liquefaction potential. Using an empirical relationship between q_T and SPT blow count, Robertson et al. (1983) extended the procedure based on SPT for evaluation of liquefaction potential to include cone penetration data. Both (N_1)_oo and q_T weakly correlate to the shear wave velocity, V_s, through site and soil specific relationships. Thus it has been argued (Bierschwale and Stokoe, 1984; Robertson et al., 1992) that the shear wave velocity can also be used as an index of liquefaction resistance.

Both CPTU and SPT induce very large shear strains in the soil around the probe (van den Berg, 1994). In comparison, the shear strain in a downhole seismic test for measuring the shear wave velocity is much smaller: usually less than 10^4%. The liquefaction phenomenon, which involves generation of excess pore water pressure of a magnitude comparable to the confining pressure, occurs at shear strains of about 0.01 to 1%. Since soil behavior depends on the strain level to a great extent, an ideal index test of liquefaction resistance should impart shear strains of similar magnitude in the medium. However, field performance of the large strain measurements (e.g., (N_1)_oo and q_T) as indicators of liquefaction resistance has been quite satisfactory. Therefore, to establish the validity of a small strain property as an index of liquefaction resistance, one needs to examine whether the statistical behavior of the small and large strain measurements are similar in a natural geologic environment. Such an exercise is undertaken in this paper.
Data over a wide strain range can be obtained from a self-boring pressuremeter test (SBPMT). Therefore, it appears logical to use the data from this test to evaluate liquefaction potential. To examine whether this approach is tenable, the statistical behavior of the self-boring pressuremeter data was compared with that of CPTU.

Description of Sites

The data used in this study come from tests carried out at three sites in the site characterization activity of the Canadian Liquefaction Experiment (CANLEX) Project. The first site near Fort McMurray, Alberta is in a tailings dam operated by Syncrude Canada Limited. The tailings are deposited inside a compacted perimeter dyke by hydraulic means. The coarse tailings are the first to settle out near the point of discharge forming a beach above water. This mode of deposition leads to the formation of layers of medium to dense sand ($D_{50} = 0.15$ mm). The target zone at this location was between 28 and 40 m depth and comprises primarily of beach deposits. The water table was at a depth of 21 m. For more details about the site refer to Sobkowicz and Handford (1990). Near the tailings dam, at a location called J-Pit, an earth embankment was built on a very loose foundation to conduct a field test on static liquefaction in Phase III of CANLEX Project. The foundation was carefully constructed by under water deposition of Syncrude Sand in the loosest possible state. Data from this site will be used in an illustrative example later in this paper.

The other two sites are situated in the Fraser River Delta near Vancouver, BC. The general stratigraphy at Massey Tunnel comprises of layers of clean loose sand between 6 m and 15 m depth. The depth of water table is approximately 2.5 m. The zone of interest at this location extends from a depth of 7 m to 15 m. At KIDD # 2, below a 1.5 m thick desiccated silt layer near surface, lies a unit of silty fine sand which grades to sand at a depth of about 6 to 8 m. Below the silty sand clean sand is found. The relative density of the clean sand layer ranges between 30 and 90%. The data used in this study from this site pertains to the layers between 7 and 20 m depth. A more detailed description of the near surface geology of Massey Tunnel and KIDD # 2 has been given by Monahan et al. (1995). In the illustrative example of this paper, a cavity expansion test performed at Massey Tunnel will be analyzed.

The In-Situ Testing Program

A total of four CPTUs with seismic measurements were carried out at Syncrude tailings dam in the detailed site characterization activity in Phase I of the CANLEX Project. In the seismic measurements, a hammer and shear beam at surface was used as the seismic source. In two CPTUs an accelerometer was
used as the receiver. In the other tests, a seismometer was used. In the SBPMT a probe with a central jetting system was used. In-situ cavity expansion tests were performed at three depths in this sounding. For the inflation of the probe in the layer of interest, bottled nitrogen at high pressure was used. The results from CPTU sounding indicates a uniform free draining cohesionless material in the target zone. These data show a fairly constant value of cone bearing varying between 12 and 16 MPa, and a fairly constant friction ratio of about 0.8 %. The shear wave velocities range between 212 and 298 m/s. Campanella et al. (1995) give a more detailed description of this testing program.

At Massey Tunnel, seven seismic piezocone penetration tests (SCPTU) and two SBPMTs were performed. At the KIDD # 2 site, eight SCPTUs and two SBPMTs were carried out. In the seismic measurements at Massey Tunnel and KIDD # 2, beam and hammer was used as the source of shear wave and an accelerometer was used as the receiver. A number of cavity expansion tests were performed at various depths in each of the four self-boring pressuremeter tests at these two sites. The self-boring pressuremeter used at Massey Tunnel employed a central jetting system while a shower head system was used at KIDD # 2. For inflation of the probe, compressed air was used. The piezocone penetration tests indicate a free draining cohesionless soil in the target zone at both the sites. At Massey Tunnel the cone tip resistance was between 4.5 and 9.0 MPa and a fairly constant friction ratio of 0.4 % was observed. The shear wave velocities varied between 150 and 220 m/s. The cone tip resistance was between 2 and 20 MPa and a uniform friction ratio of 0.25 % was measured at KIDD # 2. The shear wave velocities ranged between 150 and 230 m/s. More details regarding this testing program can be found in Reports on Activities 3A and 3B of the CANLEX Project (In-Situ Testing Group, UBC, 1995a, 1995b).

Relationship Between $G_{max}$ and $q_T$

Fig. 1 presents a scatter plot between the cone tip resistance and the small strain shear modulus at the sites described above. The values of $G_{max}$ presented in the figure were estimated from shear wave velocities from downhole SCPTU measurements using the relationship $G_{max} = \rho V_s^2$ and an appropriate value of the total mass density, $\rho$, of the medium. To minimize the effects of density on the $G_{max}$-$q_T$ relationship, data from layers at similar densities were only used. The frequency distributions of the relative densities estimated from SCPTU data following Robertson and Campanella (1986) are also shown in Fig. 1. The statistics of linear regression between log $q_T$ and log $G_{max}$ indicate that the correlation is poor for the all the sites (Table 1).

Significant scatter has also been reported by other researchers who examined similar data (e.g., Rix and Stokoe, 1991; Lee, 1992). It is therefore apparent that small strain properties do not correlate to large strain measurements.
Fig. 1. Relationship of Cone Tip Resistance and Small Strain Shear Moduli

precisely. Since the large strain measurements - q_T or (N_i)_{60} - correlate with the resistance of soils to liquefaction, a small strain property (V_S or G_max) may not be an appropriate indicator of liquefaction resistance.

Variability in Self-boring Pressuremeter Test Data

The data used in this exercise are plotted in Fig. 2. The effective cavity pressure in an SBPMT is calculated by subtracting the ambient pore water pressure from the gas pressure inside the cavity. The cavity strain, \( \epsilon_e \), is equal to the radial deformation divided by the original radius of the cavity. The cone

| Site          | Site log q_T = | # of Obs | \( r^2 \) | | \( t \) | tabled t_{0.05} |
|---------------|----------------|----------|----------|----------|------------------|
| Massey & KIDD # 2 | 0.832 log G_max - 0.697 | 53      | 0.337    | 5.09      |                  |
| Massey        | 0.780 log G_max - 0.608 | 32      | 0.301    | 3.60      | 2.042            |
| KIDD # 2      | 0.620 log G_max - 0.295 | 21      | 0.096    | 1.40      | 2.093            |
| Syncrude      | 0.227 log G_max + 0.765 | 35      | 0.035    | 1.09      | 2.042            |

Note: If |t| exceeds the tabulated values of two-tailed t distribution, the null hypothesis can be rejected, i.e., the coefficients of log G_max are significant to confidence levels better than 5%.

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tip resistance from CPTUs carried out adjacent to the locations of the SBPMTs are also shown in the figure.

For simplicity, we assume that the spread, i.e., the difference between the maximum and the minimum observed value, in a measured quantity is a good statistical representation of its variability. To establish the similarity (or
contrast) in the statistical behavior of the self-boring pressuremeter data and the cone tip resistance, we proceed as follows.

- Existence of a qualitative relationship between the pressure-expansion response in an SBPMT and cone tip resistance can be inferred from the data because the soil layers with larger cone tip resistance also show stiffer pressure-expansion response at relatively large values of $\varepsilon_s$.
- We know in addition that the large strain measurement, $q_T$, exhibit a finite variability. Thus, for the pressure expansion curves to have a similar statistical characteristic, the spread in the latter must also be finite at very large strains. To check for this, we plot the observed spread in pressure expansion curves versus $\varepsilon_s$ (Fig. 3). This plot clearly shows that the spread in the pressure expansion response increases quickly to a finite value at $\varepsilon_s = 2.5$ % and remains largely unchanged thereafter.

Therefore it can be concluded that the statistical behavior of the pressure expansion response in an SBPMT and the measured cone tip resistance is quite similar for a cohesionless geologic medium. Thus, like CPTU, the self-boring pressuremeter test may provide useful information in liquefaction studies. A simple procedure for estimation of liquefaction related soil properties from the pressure expansion data is outlined in the following section.

**Interpretation of In-Situ Cavity Expansion Test**

The interpretation basically involves derivation of the element behavior from the cavity expansion data (Fig. 4). First, the drained response of a cylindrical cavity under increasing internal pressure is computed numerically using an appropriate constitutive model and reasonable estimates of model parameters. The response of the numerical model is compared with the measured response.
in a cavity expansion test. The model parameters are altered until a reasonable match between the model and the measured response is achieved. Once a reasonable match is achieved, the model parameters are used to compute the undrained cyclic or monotonic response of a soil element. Although computation of cyclic element response provides useful quantitative information such as number of cycles to liquefaction, in this paper we will only consider the characteristics of monotonic undrained behavior for simplicity. The undrained response is computed from the drained parameters by considering the volumetric constraint of pore water.

Typically, the undrained monotonic response of a soil element exhibits one of the following characteristics.

- For very loose sands the pore water pressure increases to a value nearly equal to the in-situ confining stress and does not decrease upon further straining. The deviator stress reaches a peak value following which it decreases continuously until it reaches a small residual value at large strains. A sample showing this type of response is susceptible to flow failure or static liquefaction.

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• Medium dense sands also generate a significant pore water pressure in undrained loading. Consequently, the deviator stress decreases to a residual value following a peak. However, upon further straining the material tends to dilate. Consequently, the pore water pressure starts to decrease and the shear strength increases. A deposit showing this type of behavior may develop significant strain in an undrained loading but a catastrophic flow failure may be precluded.

• Dense sands do not exhibit any strain softening under undrained loading.

In this case, the pore water pressure starts to decrease after reaching a relatively small positive value as the skeleton tends to dilate.

As has been mentioned earlier, the estimated model parameters can be used for computing a cyclic response. From the computed cyclic response, design parameters such as number of cycles to liquefaction can be found. The derived model parameters can also be used for computing the cyclic response of an earth structure under an arbitrary loading.

The Constitutive Model

The interpretation procedure essentially involves inverse modeling of observed response of a cylindrical cavity under increasing internal pressure in a continuum to derive the model parameters. The non-linear inverse problem is solved by manual iteration. The solution process is greatly facilitated when a constitutive model with the following desirable characteristics is adopted:

• capability to capture soil behavior reasonably using a few parameters, and

• availability of reliable a-priori estimates of the magnitudes of some model parameters and a reasonable knowledge about the bounds of the values of majority of the remaining parameters.

A constitutive model fulfilling these requirements has been developed by Byrne et al. (1995\textsuperscript{a}) and was used in analyses presented later. The constitutive model is described very briefly as follows. For more details refer to Byrne et al., (1995\textsuperscript{a}, 1995\textsuperscript{b}). The model defines stress ratio, \( \eta \), as the ratio of the maximum shear stress, \( \tau \), to the mean normal stress, \( s \). In a two dimensional loading (e.g., plane strain), \( \tau = (\sigma_1' - \sigma_3')/2 \) and \( s = (\sigma_1' + \sigma_3')/2 \), where \( \sigma_1' \) and \( \sigma_3' \) are the major and the minor principal effective stresses, respectively. For determining the magnitude of the plastic strain increment vector on the plane of maximum shear, the following relationship between the stress ratio increment, \( d\eta \), and the plastic shear strain increment, \( d\gamma_p \), is assumed:

\[
\frac{d\eta}{d\gamma_p} = K_g \left( \frac{s}{P_s} \right)^{n_p-1} \left( 1 - R_t \frac{\eta}{\eta_t} \right)^2
\]

(1)

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where $K_s^e$, $n_p$ and $R_t$ are model parameters, $P_a$ is the atmospheric pressure, and $\eta = \sin \phi'$. The peak effective stress friction angle is given by:

$$\phi' = \phi_1' - \Delta \phi \times \log[s/P_a]$$  \hspace{1cm} (2)

where $\phi_1'$ is the peak friction angle at $s = 1$ atmosphere and $\Delta \phi$ is the decrease in the friction angle for a 10-fold increase in $s$. The peak effective stress friction angle in plane strain loading is larger than the corresponding triaxial values by 4° (for loose sands) to 9° (for dense sands), Lee, 1970. Following Taylor (1948), the direction of the plastic strain increment is determined from:

$$\frac{d\varepsilon^p}{d\gamma^p} = \sin \phi_{cv} - \eta$$  \hspace{1cm} (3)

where $d\varepsilon^p$ is the incremental plastic volumetric strain and $\phi_{cv}$ is the constant volume friction angle. Upon determination of the plastic strain increment on the plane of maximum shear from Eqs. (1) and (3), the plastic strain increments in the coordinate directions are computed. For calculating the elastic strain increment for a given stress increment, two material constants are needed: the bulk modulus, $B_e$, and the shear modulus, $G_c$. The stress level dependency of these elastic moduli is modeled by the following expressions:

$$B_e = K_b^e P_a \left(s/P_a\right)^{m_e}$$

$$K_c = K_g^e P_a \left(s/P_a\right)^{n_e}$$  \hspace{1cm} (4)

where $K_b^e$, $m_e$, $K_g^e$ and $n_e$ are model parameters. The elastic and the plastic strain increments are added to compute the total strain increment.

A-Priori Information on Model Parameters

Installation related disturbance near the borehole wall in a pressuremeter test affects the reliability of the data at low strains. This renders the estimation of the elastic parameter, $K_g^e$, from the virgin pressure expansion curve of an SBPMT difficult. The problem can be avoided by using supplementary seismic information from an adjacent downhole SCPTU which gives a more precise estimate of $K_g^e$. Alternatively, the unload reload data from loops performed during an SBPMT can be used. In the analyses presented in the following section, we use downhole seismic measurements to calculate $K_g^e$. In the absence of material specific information, for loose medium and dense granular materials $K_g^e$ may be assumed to be approximately 300, 600 and 1200, respectively. The corresponding bulk modulus can be estimated assuming a value of Poisson's

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ratio between 0.11 and 0.23 (Hardin, 1978). The exponents, $n_e$ and $m_e$, are
equal to about 0.5 for a wide range of sands (Duncan et al., 1980). The
constant volume friction angle, $\phi_{cv}$, for many types of sand have been reported
in the literature (Sasitharan et al., 1994) and is known to be independent of
history of loading and sample fabric. A value of $0.5 \times (\phi_1' - \phi_{cv})$ can be assumed
for $\Delta \phi$ in the absence of material specific information. The parameter $R$, varies
from a value of 1.0 for very loose deposits to about 0.75 for dense sands. The
remaining model parameters, $K_s^e$, $n_p$ and $\phi_1'$ are estimated from fitting the
model to test data over a wide strain range, e.g., data from an SBPMT or a
laboratory element test on an undisturbed sample. Although both $K_s^e$ and $K_s^p$
increase with density, it follows from the earlier discussion on comparison of
small and large strain material properties that these parameters may not
correlate with each other well. Actually, $K_s^e$ and $K_s^p$ are affected to a different
degree by prestraining, density, ageing and fabric.

Analysis of Self-boring Pressuremeter Test

Two cavity expansion tests performed at Massey Tunnel and J-Pit were
analyzed as plane strain problems to illustrate the principles of the proposed
procedure. A finite difference computer code (FLAC version 3.2; Cundall,
1993) was used in the computation. Large strains were accommodated in the
simulation by updating the nodal coordinates during loading. The appropriate
value of $K_s^e$ was calculated from the down-hole shear wave velocities from an
adjacent SCPTU. The predicted and observed cavity response from two tests
are shown in Fig. 5. The model parameters for the simulated response are also
listed in the figure.

The derived model parameters were used to compute the expected element
behavior in an anisotropically consolidated undrained triaxial extension test (Fig.
6). Plotted in this figure is the deviator stress, $\sigma_d$, and $\Delta u/\sigma_{ec}'$ against the axial
strain, $\varepsilon_{axial}$. $\Delta u$ is the pore water pressure and $\sigma_{ec}'$ is the horizontal stress at
consolidation. Undisturbed samples were obtained from these sites for
laboratory testing. The laboratory triaxial extension test data for undisturbed
samples M94 F6 C7-A (Massey Tunnel) and FS26 C3-1 (J-Pit) are also shown
in Fig. 6. In M94 F6 C7-A and FS26 C3-1, the samples were consolidated to a
horizontal effective stress of 62 and 56 kPa, respectively. The corresponding
vertical stresses were twice those values. The observed behavior compares
reasonably with the element response predicted from the self-boring
pressuremeter tests. It appears, therefore, that in a more elaborate analysis of
an earth structure comprising of similar material, the estimates of the material
parameters can be used to compute the undrained response under an arbitrary
loading.
Summary and Conclusions

The performance of large strain tests such as SPT and CPTU in assessment of liquefaction potential is understood to be quite satisfactory. In this paper we examined whether $G_{\text{max}}$ (or $V_s$) and self-boring pressuremeter tests have relevance in liquefaction studies. To check for their feasibility, we compared the statistical behavior of $G_{\text{max}}$ (or $V_s$) and the pressure expansion curves in an SBPMT with that of cone tip resistance. Since the pressure expansion response in a self-boring pressuremeter test and the cone tip resistance exhibit similar statistical qualities, the pressuremeter test appears to have some relevance in liquefaction related studies. However, the correlation between the cone tip resistance and $G_{\text{max}}$ (or $V_s$) was found to be statistically untenable. Hence, the
relevance of the small strain shear modulus or shear wave velocity in liquefaction assessment becomes questionable. A procedure for the interpretation of self-boring pressuremeter tests to derive liquefaction related material properties was proposed. The technique was illustrated and evaluated against laboratory experiments. The results indicate that the model parameters back figured from pressuremeter data can predict the undrained monotonic response of cohesionless soils.

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(b) A copy of a paper entitled “A simple understanding of the liquefaction potential of sands from self-boring pressuremeter tests” (Hughes et al., 1997)
ABSTRACT: Although the conventional Menard pressuremeter can be used in sands, disturbance from stress relief during the formation of the pocket for the pressuremeter can significantly influence the results. However, a self-bored pressuremeter can readily be deployed in loose or dense sands in such a manner that the disturbance is limited. The data from this tool can provide some insight into the state of the deposit. Unlike cone data, which can be interpreted only in an empirical manner, pressuremeter data can be interpreted with little recourse to empiricism.

1. INTRODUCTION

Although the conventional Menard pressuremeter can be used in sands, disturbance from stress relief during the formation of the test pocket for the pressuremeter can significantly influence the results. However, a self-bored pressuremeter can readily be deployed in loose or dense sands in such a manner that the disturbance is limited. The data from this tool can provide some insight into the state of the sand. Unlike cone data, which can be interpreted only in an empirical manner, pressuremeter data can be interpreted with little recourse to empirical correlations. The usefulness of these tests in determining the state of the sands is the objective of this paper.

The Canadian Liquefaction Experiment (CANLEX) research project was set up exclusively to consider the most suitable methods of assessing the liquefaction potential of sands. In this study, six sand sites have been examined in detail. Two of these sites were at the Syncrude Canada Ltd. Mine in Northern Alberta. At the J-pit location, the sand was intentionally deposited in a loose state as a foundation for a trial embankment. The Cell 24 site was at a height of 40 m in elevation. At all of these sites, self-boring pressuremeter tests (SBPMTs) were performed. This paper outlines the salient results of pressuremeter tests at the two Syncrude sites in a very simplistic, "broad-brush" manner. From the results of this overview, the major features of the behaviour of the sand can be determined. These findings are consistent with the results of the laboratory tests conducted on samples obtained by exceedingly expensive freezing techniques.

2. SITE DESCRIPTION

Cell 24 is situated in the 40 m-high Syncrude Tailings Dam at Fort McMurray, Alberta. The tailings are essentially free-draining, angular to sub-angular fine quartz sand, obtained by extraction of crude petroleum from naturally-occurring oil sand. The tailings were deposited by hydraulic means in a settling basin inside a compacted perimeter dyke. Details on the site characterization in the test zone from 28-38 m can be found in Campanella et al. (1995). A typical cone tip profile through this cell is presented in Figure 1A.

At a nearby location, J-Pit, a very loose foundation of Syncrude tailings, approximately 12 m thick, was constructed. The foundation was carefully constructed by underwater deposition of the same sand as that used at Cell 24. On the top of this foundation, an embankment was constructed. The upstream side of the embankment was rapidly loaded with the objective of creating a static liquefaction slide through the loose foundation. Details of the field investigation at Cell 24 and J-Pit are outlined in Hoffman et al. (1996). The details of the full scale flow liquefaction tests and some of the preliminary conclusions regarding the limited movement during loading are contained in Robertson et al. (1996). The cone tip profile data, obtained at the J-Pit location CPT20, within 5 m of the pressuremeter tests, are presented in Figure 1B.

3. RELIABILITY AND REPEATABILITY OF SBPMT

The general quality of the pressuremeter data is probably best described in terms of its repeatability in a sand that is in a fairly uniform state as measured by the variation in the cone tip stress. An indication of the repeatability of the data can be seen in the pressuremeter tests at 35.0, 35.8 and 36.6 m depth, in the same hole, at cell 24, (Figure 2) and the three tests, in different holes 4 m apart, but at the same depth of 4.5 m at J-Pit (Figure 3). The SBPMTs, particularly at J-pit, lie in a very narrow band. These results are very consistent with the cone tip resistances measured at J-Pit. At Cell 24 the spread of the SBPMT data is slightly larger. However, that is consistent with the variation in the cone data.

Therefore, it would appear that the self-boring pressuremeter data is consistent with the cone data in reflecting the variation of the behaviour of the sand.

4 FUNDAMENTAL DATA PROVIDED BY SBPMT AND THE CONE

At any depth the pressuremeter test provides a complete curve of deformation behaviour, defined by (in many cases) several hundred points. Hence, the pressuremeter provides many data sets which relate the pressure to a particular strain. In contrast,
the cone at any particular depth gives only one data set relating a tip stress to some average strain. Hence, mathematically speaking, with several data sets, there is the possibility of determining the state of the sand, whereas with one data set there is only the possibility of estimating the state of the sand through empirical correlations.

Since the cone tip stress is affected by many material parameters, it is difficult to be certain of the state of the sand determined from this one number alone. For instance, sands with the same tip stress could have different densities, depending on the relative compressibility and stress history of the sand. Therefore, the cone tip stress alone, no matter how accurately it is determined, can be used to determine only an indication of the state of the sand.

5 DETERMINING THE STATE OF SAND FROM SBPMT

A simple "broad-brush" approach is adopted in this presentation to illustrate the usefulness of SBPMT. The simplest approach to infer the state of the sand is to compare the field SBPMT data to the ideal pressure-expansion curve which would be generated if the sand deformed at the critical state friction angle, \( \phi_{cv} \) (Gibson and Anderson, 1961). At this state the sand would deform in drained loading at constant volume. Hence, if the field curve is above this ideal line, then additional energy or work is required to shear the sand. In contrast, if the field curve is at or below this line, an unstable situation could occur, in which the sand could collapse on shearing.

In its simplest form, this ideal line can be generated by four simple assumptions:

1. The sand is initially in a \( k_s \) stress state.
2. Initially, shear occurs under elastic conditions until the stress ratio reaches the critical state. An average secant elastic modulus, from zero shear stress up to the stress ratio that the critical state is reached, is required. (If the data is well defined, the average elastic secant modulus is determined from the initial slope of the pressuremeter curve, or it is taken to lie between 0.3 to 0.5 times the modulus derived from the unload-reload loop).

3. At the critical state stress ratio, as defined by the critical state friction angle, \( \phi_{cv} \), the sand is assumed to shear at constant volume.
4. The deformation of the centre of the instrument is assumed to be approximated by plane strain conditions.

Applying this procedure to the SBPMT data from the two sites at Syncrude, it is clear that at the Cell 24 (Figure 2) additional energy over and above that required to shear the sand under constant volume (Poisson's ratio=0.5) is necessary to shear the soil. In contrast, the sand at 4.5 m depth at J-Pit is close to the critical state line and hence, would be very close to an unstable state (Figure 3).

With the slightly more complex model proposed by Carter et al. (1986), the effect of elastic sand compressibility on critical state is easily included by considering Poisson's ratio to be a function of compressibility. Included in Figures 2 and 3 are the ideal pressure-expansion curves for Poisson's ratio of 0.5, (the Gibson and Anderson solution) and a possible lower limit for sands of 0.0, giving a practical band for constant volume behaviour. (In compressive silts the apparent Poisson's ratio can actually be modelled by a negative value.)
6. LABORATORY TEST DATA

Several undisturbed samples extracted by ground freezing from Cell 24 and J-Pit have been tested in the laboratory (Konrad and Saint-Laurent, 1995; and Vaid et al., 1996). Figures 4 and 5 present the range of response in triaxial compression and extension tests for samples from Cell 24 (drained tests) and J-Pit (undrained), respectively. After thawing at a small confining pressure, the samples were consolidated anisotropically to the estimated in-situ state of stress.

The laboratory tests show that, in triaxial compression, both sites exhibit dilative behaviour on shearing. In contrast, in extension tests the Cell 24 data shows a tendency to dilate after an initial contraction, whereas at J-Pit the results are much less certain and are possibly representative of a material which is close to deforming at a constant volume.

In these natural samples the in-situ fabric will have been retained. As a result of the vertical deposition, there is a significantly greater grain contact in the vertical direction rather than the horizontal direction. In general, the volumetric behaviour of granular material is dominated by the relative direction of the major principal stress to the direction deposition.

Hence, for tests in triaxial compression, in which the major principal stress and the direction of deposition coincide, the dilation effects are larger than for extension tests, in which the principal stress is normal to the direction of deposition (Vaid et al. 1990). This behaviour is clearly evident with the tests at the 35m level at Cell 24. The volumetric strain in triaxial compression is far greater than for extension. However, after about 2.5% strain, both tests exhibit a positive dilative behaviour.

In a self-boring pressuremeter test the shear planes are vertical, normal to the direction of grain deposition. Hence, the triaxial extension (rather than triaxial compression) test data is more indicative of the behaviour of sand tested by the SBPMT.

7. CONCLUSIONS

With the use of a very simple model, a clear qualitative indication of the state of the sand can be obtained from a visual inspection of the relative position of the field pressuremeter curve to the constant volume pressure-expansion curve. With a slightly more complex model, with only one more variable, the dilation rate (v), the distance from the critical state line to the field curve can be defined in terms of the volume change which is required to shear the soil under drained conditions (Hughes et al., 1977). This, as pointed out by Yu et al. (1994), can then be directly related to the state parameter.

The general "broad brush" approach to the determination of the state of the sand, as to whether it was in a dilative or contractive state is in agreement with the laboratory results and was obtained at a fraction of the cost. For projects of larger areas, a few well-done self-boring pressuremeter tests could provide the reference data needed to develop empirical correlations with cone data at the SBPMT locations. In this way, the more productive CPT could be used throughout the site to provide stratigraphic detail and indicate locations of concern or where additional testing and analyses would be required.

8. ACKNOWLEDGEMENTS

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APPENDIX P

Phase III Full Scale Liquefaction Field Test
a) A copy of a paper entitled "CANLEX Phase III full scale flow liquefaction test: planning, objectives and conclusions" (Robertson et al., 1996)
CANLEX Phase III Full Scale Flow Liquefaction Test: Planning, Objectives and Conclusions

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ABSTRACT
A full scale liquefaction test was carried out at the Syncrude Canada Site near Fort McMurray, Alberta during the summer of 1995. The liquefaction test involved the rapid loading of loose, saturated sand in an attempt to statically trigger a flow slide by flow liquefaction. The design of the full scale test was based on previous field experience, centrifuge testing and numerical modeling. The field test site was constructed by first hydraulically placing about 10 m of sand into standing water, contained within an abandoned borrow pit (J-Pit). Construction of the loose, saturated sand foundation was followed by the slow drained construction of an overlying clay containment dyke around part of the perimeter of the borrow pit and the subsequent rapid infilling of the containment area with tailings. This paper presents an overview of the planning, objectives and conclusions for the full scale flow liquefaction test associated with Phase III of the CANLEX Project.
INTRODUCTION

The Canadian Liquefaction Experiment (CANLEX) involves the characterization of loose sand in order to predict its liquefaction response. Several test sites have had detailed site characterization carried out to better understand the response of the sand to various types of loading. This characterization has involved extensive in-situ testing and sampling as well as detailed laboratory testing. However, to fully understand the response of loose sand to certain types of loading it was necessary to carry out a full scale test. A full scale test was carried out at the Syncrude Canada site near Fort McMurray in Alberta during the summer of 1995.

The purpose of the full scale test was to evaluate our ability to predict liquefaction phenomena. In dealing with liquefaction there are essentially two main concerns:

1. What level of disturbance is required to trigger soil liquefaction? and
2. What are the likely deformations?

The triggering of cyclic liquefaction under undrained cyclic loading conditions is reasonably well understood. Basically, the soil has a resistance to cyclic loading that is primarily a function of the soil density and grain characteristics. If the imposed cyclic stresses from an earthquake or other sources exceeds the cyclic resistance then cyclic liquefaction is triggered. Deformations can be very large but essentially stop when the cyclic loading stops.

Triggering of flow liquefaction under static (monotonic) loading is associated with a strain softening material response. A flow slide may occur if the soil can be triggered to strain soften and if the driving shear stresses are larger than the residual undrained shear strength of the soil. For a flow slide to develop, there must be a sufficient volume of strain softening soil present in the loaded areas and a ground geometry to allow a kinematically admissible mechanism to form.

For the full scale liquefaction test, the triggering of soil liquefaction can be achieved with either a static or a dynamic trigger. While the cyclic trigger is quite well understood, there is considerable uncertainty concerning the static trigger.

LIQUEFACTION FUNDAMENTALS

Liquefaction can occur in soils and refers to a sudden large increase in deformations associated with a large decrease in stiffness and/or strength. The following definitions were developed by CANLEX (Robertson et al. 1994):

Flow Liquefaction
Applies to strain softening soils only.

- Requires a strain softening response in undrained loading resulting in constant shear stress and effective stress, as illustrated in Figure 1.
- Requires in-situ shear stresses greater than the ultimate or minimum undrained shear strength.
- Flow liquefaction can be triggered by either monotonic or cyclic loading.
• For failure of a soil structure to occur, such as a slope, a sufficient volume of material must strain soften. The resulting failure can be a slide or a flow depending on the material characteristics and ground geometry. The resulting movements are due to internal causes and can occur after the trigger mechanism occurs.
• Can occur in any metastable saturated soil, such as very loose granular deposits, very sensitive clays and loess (silt) deposits.

Cyclic Liquefaction
Applies to strain softening and strain hardening soils.
• Requires undrained cyclic loading during which shear stress reversal occurs or zero shear stress can develop (i.e. where in-situ static shear stresses are low compared to cyclic shear stresses), as illustrated in Figure 2.
• Requires sufficient undrained cyclic loading to allow effective stress to reach essentially zero.
• At the point of zero effective stress no shear stress exists. When shear stress is applied, pore pressure drops and a very soft initial stress strain response can develop resulting in large deformations.
• Deformations during cyclic loading can be large as the effective stress passes through the zero condition, but deformations generally stabilize when cyclic loading stops. The resulting movements are due to external causes and occur only during the cyclic loading.
• Can occur in almost all sands provided the cyclic loading is sufficiently large in size and duration.
• Clayey soils can experience cyclic liquefaction but deformations are generally small because the zero effective stress state is generally not reached and some cohesive strength maybe present. Deformations in clay soils are often controlled by rate effects (creep).

The above phenomena for flow liquefaction is complicated by the fact that the undrained response of soil is generally anisotropic or load path dependent. Hence, although the soil may be strain softening if loaded in one direction, it may be strain hardening if loaded in another direction. Different parts of a soil slope may be failing under different load paths and the combined resistance along a potential failure zone in the slope must be strain softening for a flow slide to occur.

Dense soil is generally strain hardening and cannot experience flow liquefaction. However, sand can be triggered to experience cyclic liquefaction under cyclic loading, as illustrated in Figure 2. The strains will be very small until the stress point reaches the phase transformation line and moves toward the point of zero effective stress on unloading. Cyclic loading that causes the stress point to repeatedly pass through the zero state point can lead to very large cyclic strains. Field observations after major earthquakes indicate that large displacements due to cyclic liquefaction can occur on very flat slopes and such movements are generally referred to as lateral spreading.

In order to obtain a better understanding of both static an dynamic triggers, two field tests would be required.

Test 1: Flow Slide
This would involve loose sand and a relatively steep slope such that triggering will cause a flow slide. The trigger could be either static or dynamic.
Test 2: Lateral Spread
This would involve either loose or dense sand and a more moderate slope such that triggering would result in a lateral spread rather than a flow slide. In this case the trigger must be dynamic (cyclic).

Static triggers could involve:
- Increasing the driving shear stresses by loading the crest or unloading the face of a slope, as illustrated in Figure 3 (stress path A-B on Fig. 3b).
- Reducing the normal effective stresses by increasing the pore water pressures, as illustrated in Figure 3 (stress path A-C on Fig. 3b).

The concept in either case is to drive the stress point into the potentially unstable region. In practice this can be achieved by: (a) placement of a load at the crest of the slope; (b) steepening the slope or excavating the toe; (c) removing water load from the face of the slope (drawdown); and (d) increasing the pore water pressure within the slope (no water on face). This latter case was used by Eckersley (1986) on small scale model tests.

A variety of potential dynamic triggers were considered: (a) impact loading: dropping a large weight, similar to dynamic compaction; (b) blasting and (c) vibroseis: high frequency vibration.

Previous Flow Liquefaction Experience

Field experience from a number of flow slides at the Suncor Site in Alberta were examined (Plewes et al., 1989). A number of flow slides were initiated by the rapid construction of tailings embankments over loose tailings sands in the 1970's. The experience indicated that the placement method, rate of construction and high static shear stresses were key factors in the initiation of liquefaction. Experience also indicated that the hydraulic transport and placement of tailings sand under water leads to the loosest in-situ state for freshly deposited tailings sand. The rate of embankment placement was also very important. Rates in excess of about 0.3 m per day could lead to flow liquefaction failures, while slower rates resulted in stable conditions. This indicates that drainage is a key factor in curtailing the excess porewater pressures resulting from the applied embankment loading and preventing the triggering of flow liquefaction.

A number of centrifuge tests were carried out to simulate the rapid placement of load on a loose foundation sand layer (Byrne et al., 1995). These tests indicated that a flow liquefaction failure could be triggered if the (Syncrude) sand layer had a relative density of less than about 35%.

Laboratory model tests conducted by Eckersley (1986) indicated that flow liquefaction can also be triggered by raising the boundary water level. Once triggered, the subsequent movements occurred rapidly under essentially undrained conditions causing porewater pressures to rise further and the resistance of the soil to drop. This type of loading has the advantage that the rate of loading is not a factor, since flow liquefaction is triggered by a decrease in effective stress due to the rising water level.
FULL SCALE FLOW LIQUEFACTION TEST

A number of factors influenced the ultimate design of the full scale field test: safety to life, safety to equipment, construction procedures compatible with Syncrude procedures, costs, timing, feasibility, predictability, reliability, field and model test experience, and numerical simulation. Based on these considerations, it was agreed that the full scale test should investigate a flow slide and that the trigger should be static comprising both rapid loading as well as boundary water level loading.

The field experience at the Suncor site in the 1970's (Plewes et al., 1989) together with the centrifuge testing and numerical analyses (Byrne et al. 1995) suggested that a minimum embankment height of about 8 m would be required to trigger flow liquefaction and a flow slide. A target height of 10 m was desirable with an ability to go to 15 m. However, based on safety concerns and cost, the maximum height of the embankment was limited to only 8 m.

The test was carried out in J-Pit on the Syncrude Canada Ltd. site near Fort McMurray, Alberta in the summer of 1995. J-Pit is an old borrow pit located north of the Mildred Lake Settling Basin. The pit is approximately 400 m long by 300 m wide and 10 m deep, and was full of water prior to sand placement. The test was carried out in the northwest corner of J-Pit, as shown in Figure 4. The pit was filled with tailings sand placed mostly under water. This sand will be referred to as beach below water (BBW). Sand was placed to form a working platform at elevation 321 m. The upper 3 m of sand was placed above water and is referred to as beach above water (BAW). A gently sloping beach down to the water level at elevation 318 m was formed. The elevation of 318 m was controlled by the outlet for J-Pit which is located in the southeast corner.

The objective of the test was to rapidly load the loose sand. To do this, a containment cell was required to allow for the rapid, controlled loading. The containment cell was comprised of a clay embankment carefully constructed over the loose sand and a hydraulically placed sand containment structure placed on natural ground. The rapid loading was carried out in an effort to trigger flow liquefaction within the loose foundation sand. The clay embankment was constructed across the mouth of a bay in the northwest corner of J-Pit. The clay embankment was constructed to a total height of 8 m with side slopes of about 2.5 horizontal to 1 vertical. To ensure safety during construction and subsequent tailings sand placement, the embankment was made from compacted clay. The rapidly placed tailings sand was contained between the clay embankment and a compacted hydraulically constructed sand structure, as shown on Figure 4. The compacted sand structure was located predominantly over natural ground. The tailings sand was placed hydraulically so that the soil structure could be rapidly loaded as well as increasing the water loading. The water caused seepage under the embankment.

The sequence of construction was as follows;
1. Tailings sand was placed predominantly below water to fill J-Pit.
2. An 8 m high clay embankment was constructed slowly to achieve essentially drained response under close supervision as the initial part of the development of a loading cell.
3. A compacted sand structure was constructed around the back of the clay embankment to form the enclosed cell. The enclosed cell was about 200 m by 100 m and 8 m deep.
Tailings sand was rapidly placed hydraulically into the enclosed cell. An overflow weir was placed on the clay embankment in the north corner of the enclosed area to allow release of excess water into J-Pit.

The rate of loading was such that the enclosed area was full of water and some sand within 12 hours and full of sand within 36 hours.

Before the full scale test the loose foundation sand was fully characterized using extensive in-situ testing and sampling. Full details of the site characterization carried out at the site before the full scale test are given in a companion paper by Hofmann et al. (1996a). Extensive instrumentation was installed to record the response of the loose foundation sand to the loading and to record deformations. Details of the instrumentation, construction and observed response are given in a companion paper by Hofmann et al. (1996b). Numerical modeling was carried out to predict the response of the structure to the rapid loading and to assist in the analyses of the results. Details of the numerical analyses are contained in companion papers by Byrne et al. (1996); Soroush et al., (1996) and Cathro and Gu (1996).

In general, the observed response to the rapid loading was a pore pressure profile consistent with the establishment of steady state seepage and displacements less than 0.05 m.

LESSES LEARNED

A flow slide was not initiated during the rapid loading in 1995. The analyses indicate that the height of the embankment and the type of loading was not sufficient to trigger flow liquefaction within the loose sand foundation. The test was successful in that a full scale loading of an embankment was carried out over sand that was considered very loose using conventional techniques. The analyses has also illustrated the difficulty in modeling flow liquefaction. The methods used were the state-of-art methods not state-of-practice. New developments were made in the modeling of flow liquefaction to include the strain softening soil response. The details of each method were different but many features were similar. Strain softening soil response is very difficult to correctly model due to possible non uniform strain distribution. Strain softening soil response requires a strain compatible technique, such as finite elements, rather than the more traditional limit equilibrium techniques.

The results of laboratory testing carried out after the full scale field test on both undisturbed and reconstituted samples of the sand placed in J-Pit have shown that the sand is slightly strain softening in undrained triaxial compression loading but highly strain softening in undrained triaxial extension loading. The response in undrained simple shear loading is close to that of extensional loading. The undrained shear strength ratio $\left(\frac{\sigma_u}{\sigma_v}\right)$ based on undisturbed samples was approximately 0.30 in triaxial compression and approximately 0.03 in triaxial extension. Figure 5 shows results of hollow cylinder torsion tests carried out on loose water pluviated reconstituted samples of Syncrude sand consolidated to the same void ratio as measured in-situ and loaded under different directions of loading. It is clear that the undrained shear strength in triaxial extension is significantly smaller than in compression. Figure 5 also shows the results of a torsional simple shear test where the
principle stresses are rotated from compression to simple shear ($\alpha_c = 45^\circ$). In this case the material strain softens directly. This illustrates the potentially metastable state of the loose sand in J-Pit when the principle stresses rotate.

The configuration of the full scale test for CANLEX was dominated by predominantly compression type loading with very little rotation of principle stresses. This was due to the geometry of loading from the embankment. It is now clear that if the loading had contained a greater proportion of simple shear and extensional type loading that flow liquefaction would have resulted. The geometry of loading selected was based on the Suncor experience, centrifuge testing and numerical modelling, as well as by safety and cost issues. An important lesson learned from this full scale field test is the importance of the direction of loading in undrained soil response. Simple state-of-practice design charts that predict in-situ soil state should be used with caution since they may not reflect the large variation in soil response due to direction of loading.

**SUMMARY**

A full scale liquefaction test was carried out at the Syncrude Canada site near Fort McMurray during the summer of 1995. The objective of the test was to construct an embankment over loose saturated sand and then to rapidly load the embankment by hydraulically placing sand tailings behind the embankment. The rapid loading was carried out in an effort to trigger flow liquefaction within the loose foundation sand. Site characterization was carried out before and after construction of the clay embankment to evaluate the in-situ state and response of the loose foundation sand. Instrumentation was installed to monitor the response of the sand and the deformation of the embankment.

A flow slide was not initiated during the rapid loading in 1995. The analyses indicates that the height of the embankment and the type (direction) of loading was not sufficient to trigger flow liquefaction within the loose sand foundation. However, the test was successful in that a full scale loading of an embankment was carried out over sand that was considered very loose using conventional techniques. Details of the site characterization, instrumentation, monitoring and analyses are given in companion papers. An important lesson learned from this full scale field test is the importance of the direction of loading in undrained soil response.

Simple state-of-practice design charts that predict in-situ state should be used with caution since they may not reflect the large variation in soil response due to direction of loading.

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Klohn-Crippen Consultants Ltd., AGRA Earth and Environmental Limited, Golder Associates Ltd., and Thurber Engineering Ltd., as well as faculty and students from the Universities of Alberta, British Columbia, Laval and Carleton.

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Figure 1. Monotonic and cyclic undrained behavior of loose sands.

Figure 2. Cyclic behavior of sands and cyclic liquefaction.

MONOTONIC UNDRAINED BEHAVIOR

Void ratio ε

Mean effective stress, p'

SS
LSS
SH
USL (Ultimate State Line)

Strain softening (SS)

Phase Transformation
Collapse Instability

Peak strength envelope

Mean effective stress, p'

Cyclic Liquefaction

No. of cycles, N

CYCLIC LIQUEFACTION

Note: SS: Strain softening response
SH: Strain hardening response
LSS: Limited strain softening response
qST: Static gravitational shear stress
SU: Ultimate undrained shear strength
US: Ultimate State
Figure 3. Possible triggering mechanisms for flow liquefaction (a) mechanisms in the field, (b) Stress paths.
Figure 4. Plan view and cross-section of full scale flow liquefaction test at J-Pit, Syncrude Canada Ltd. site in Ft. McMurray.
Figure 5. Undrained hollow cylinder torsion tests on loose Syncrude sand (Vaid et al., 1996).
b) A copy of a paper entitled “CANLEX Phase III full scale flow liquefaction test: site characterization” (Hofmann et al., 1996a)
CANLEX PHASE III
FULL SCALE LIQUEFACTION TEST: SITE CHARACTERIZATION

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ABSTRACT

The primary objective of Phase III of the Canadian Liquefaction Experiment (CANLEX) was to statically trigger flow liquefaction of a loose, saturated sand deposit. The first step towards meeting this objective was to construct a loose sand deposit, by hydraulically placing 10 m of sand into standing water contained within an abandoned borrow pit (J-pit) at Syncrude Canada Ltd. Once the foundation had been placed, an investigation was undertaken to characterize the sand and confirm that there was a potential for triggering flow liquefaction. The information obtained during site characterization was also used to select the optimum locations for instrumentation and placement of the containment dykes. The in-situ testing techniques and sampling methods utilized for site characterization during Phase III of the CANLEX project are described, including Cone Penetration Testing, Seismic Cone Penetration Testing, Standard Penetration Testing, Geophysical Logging, Self Boring Pressuremeter Testing, and in-situ ground freezing to obtain undisturbed samples of loose sand.

RÉSUMÉ

L'objectif principal de la Phase III de l'Expérience Canadienne sur la Liquefaction (CANLEX) était de provoquer de façon statique l'écoulement par liquéfaction d'un dépôt de sable lâche saturé. La première étape comportait la construction d'un dépôt de sable lâche en plaçant hydrauliquement 10 m de sable dans une eau stagnante à l'intérieur d'une carrière d'emprunt abandonnée (de type J) sur un site de la Syncrude Canada Ltd. Après avoir placé la fondation, une étude a été effectuée pour caractériser le sable et pour vérifier la possibilité du

INTRODUCTION

The density of the hydraulically placed sand deposit in J-Pit was first characterized by conducting 26 Cone Penetration Tests (CPT) in an 80 m by 80 m grid across the site. Figure 1 shows the locations of the CPTs. Based on the CPT screening, the location of the embankment was selected, considering both the location of the loosest sand and the geometry of J-Pit with respect to adequate runout distances for the static liquefaction test. The location of an 8 m radius circle near the south end of the clay embankment footprint was also selected for detailed characterization, including in-situ testing and undisturbed sampling utilizing in-situ ground freezing. All testing and sampling was carried out within a target zone located between 3 and 7 m below the ground surface.

The in-situ testing carried out around the 8 m radius circle included: Standard Penetration tests (SPT), Dynamic Cone Penetration Tests (DCPT), Cone Penetration Tests with pore pressure measurements (CPTu), Seismic Cone Penetration Tests with pore pressure measurements (SCPTu), Plate Load Tests, Self-Boring Pressuremeter Tests (SBPMT) and Geophysical Logging (GEO). In-situ ground freezing and undisturbed sampling were carried out in the center of the circle within a 1 m radius. Figure 2 shows the 8 m radius circle where detailed characterization was carried out. This paper describes the various techniques utilized to characterized the loose sand deposit constructed in J-Pit and presents a brief summary of the information obtained.

CONE PENETRATION TESTING (CPT, CPTu AND SCPTu)

Cone Penetration Tests (CPT and CPTu) were carried out by Conetec Investigations Ltd. of Vancouver, British Columbia using an integrated electronic cone system. The cone had a tip area of 10 cm² and a sleeve friction area of 150 cm². During penetration, the tip resistance, \( q_c \), Sleeve Friction, \( f_s \), and Penetration Pore Pressure, \( u_t \), were recorded at 5 cm depth intervals. In addition, pore pressures were recorded during pauses in penetration. The cone was pushed using a 25 ton track-mounted cone penetration testing unit supplied by Conetec Investigations.

Shear wave velocity measurements (SCPTu) were also taken at CPT locations CPT11, CPT14 and CPT26. Each time cone penetration was halted to add additional push rods. Before taking shear wave trace measurements, the rods were decoupled from the pushing ram to minimize background noise. Measurements were made at 1.0 m intervals.
A total of 22 cone penetration tests and 4 seismic cone penetration tests were performed at the CANLEX test site, at the locations shown in Figure 1, prior to construction of the containment dyke. The CPT screening indicated that the loosest sand was located at the southeast corner of the clay embankment footprint. The CPT profiles established in the vicinity of the detailed characterization zone, including the east end of Line 5 where the lowest cone tip resistances were encountered, are shown in Figure 3a. The average normalized cone tip resistance, $q_{ct}$, was 2.35 MPa in the target zone. Shear wave velocity measurements, taken during seismic cone penetration tests, had an average normalized value, $V_{sl}$, of approximately 127 m/s in the target zone, as shown in Figure 3b.

CPTs were also conducted in two boreholes after construction of the containment dyke to determine whether or not drained construction of the overlying clay dyke had caused any densification of the hydraulically placed sand foundation. The boreholes were located along the crest of the 8 m high containment dyke, which were then drilled out down to the top of the foundation sand. CPTs were then conducted approximately 12 m into the sand deposit. The CPTs conducted after embankment construction indicated no appreciable change in the density of the sand foundation.

**STANDARD PENETRATION AND DYNAMIC CONE PENETRATION TESTS**

Standard Penetration Tests (SPT) were performed in three boreholes and a Dynamic Cone Penetration Test (DCPT) was performed in one borehole located, as shown in Figure 2. The boreholes where SPTs were conducted were advanced using a mud rotary technique with an 82.6 carbide bit and 44.5 mm diameter AW drill rods. Tests were performed in the target zone at approximately 0.75 m intervals. Quick gel (Bentonite) was used as a slurry mud for Boreholes SPT 1, SPT 2 and SPT 3. A standard 63.5 kg safety hammer was dropped 760 mm using an electric winch. A split spoon which did not require a liner was used for sampling.

Energy measurements were recorded at several depths in SPT 4 and it was estimated that the average energy transferred was approximately 55%. All SPT measurements were corrected for energy and overburden stress. The normalized $(N_1)_{60}$ values obtained from boreholes SPT 1, SPT 2 and SPT 3 are summarized in Figure 3c. The average $(N_1)_{60}$ in the target zone was approximately 3.4 blows per 300 mm of penetration.

The borehole for the DCPT was advanced using an auger drilling rig and Hivis Polymer drilling mud. A 57.2 mm diameter cone, with a 60° apex angle and a 12.5 mm shoulder was used with 44.5 mm diameter AW drill rods. The $N_{DCPT}$ values varied with depth throughout the target zone from 1 to 6 blows per 300 mm of penetration.

**GEOPHYSICAL LOGGING**

Geophysical logging was carried out at two borehole locations within the 8 m radius circle to provide detailed logs of interpreted density, void ratio and moisture content with depth. Mud
rotary methods were utilized with a 14.3 cm diameter drill bit, using a track mounted drilling rig. A mixture of high yield Wyoming bentonites known as Quik Gel and XL Ultra Servis was used with a mud density of approximately 1.4 g/cm$^3$, to maintain borehole wall stability.

Century Geophysics Canada Corp. carried out the geophysical logging using two tools: a 9071A compensated Neutron Tool and a 9036AA Compensated Density Tool. Logging was carried out as the tool was raised from the bottom of a mudded borehole at a rate of 6 m/minute. Figure 4 shows the void ratio profile established based on geophysical logging undertaken in Boreholes GEO1 and GEO2.

**SELF-BORED PRESSUREMETER TESTS**

Self-bored pressuremeter tests were performed by Hughes Insitu Engineering Inc., at three borehole locations around the 8 m radius zone of detailed characterization, as shown in Figure 2. Pressure-expansion curves were obtained in the loose sand foundation at approximately 0.8 m intervals through the target zone. The tests were stress-controlled by regulating the pressure from a nitrogen bottle to the instrument. All pressure-expansion curves were corrected for membrane stiffness and system compliance.

The softest response encountered was noted at a depth of approximately 4.53 m in all three boreholes in the sand foundation. The slopes of the unload-reload loops in each borehole at this depth gave a shear modulus of approximately 7,200 kPa. Figure 5 shows the pressure-expansion curve obtained at a depth of 4.8 m in Borehole SBPMT2. Based on comparison with an elasto-plastic model for material failing at a constant friction angle with no dilation, it was estimated that the sand at approximately 4.53 m was very close to the critical condition at which shear would occur under constant volume.

**PLATE LOAD TESTS**

Rapid downhole plate load tests were carried out in several boreholes located near the 8 m radius circle to investigate the potential for using such tests for estimating the undrained shear strength of loose sand deposits. The results from this work are presented in a complementary paper given at this conference by Fear, et al. (1996).

**IN-SITU GROUND FREEZING AND UNDISTURBED SAMPLING**

In-situ ground freezing, utilizing liquid nitrogen, was used to freeze a 1 m radius column of sand in the center of the 8 m radius detailed characterization zone for obtaining highly undisturbed samples of sand. Once freezing was complete, undisturbed sampling was carried out with 100 mm and 200 mm diameter Cold Regions Research Engineering Laboratory (CRREL) barrels. In total, 6.4 m of 200 mm diameter undisturbed sand core and 3.9 m of 100 mm diameter core was recovered from within the target zone for void ratio determination and laboratory testing. The
range in void ratio estimated from geophysical logging compares well with the range in void ratio
determined based on trimmings from the frozen sand core samples, as shown in Figure 4. This
undisturbed sampling technique is described in detail by Hofmann, et al. (1995).

CONCLUSIONS

The results from the site investigation appear to indicate that the sand deposit formed by
hydraulically placing tailings in J-pit was loose in nature. Low normalized SPT blowcounts
\((N_1)_{60}\) of 3.4, low normalized CPT penetration resistances \((q_{cl})\) of 2.35 MPa and low
normalized shear wave velocity measurements \((v_s)\) of 127 m/s were recorded in the target zone
from 3 to 7 m below the surface of the loose sand foundation.

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collaboration includes the geotechnical consultants, EBA Engineering Consultants Ltd.,
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Figure 1: Plan View of Embankment showing Locations of Cone Penetration Tests.

Figure 2: 8 m Radius Detailed Characterization and Sampling Zone
Figure 3a: Cone Penetration Test Profiles Obtained in and near Detailed Characterization Zone.

Figure 3b: Shear Wave Velocity Profiles obtained during Seismic Cone Penetration tests.

Figure 3c: Standard Penetration Test Profiles obtained in Detailed Characterization Zone.
Figure 4: Void Ratio Estimates Determined from Geophysical Logging and Trimmings from In-situ Frozen Core Samples.

Figure 5: Self-Boried Pressuremeter Test Result obtained from Borehole SBPMT2 in Detailed Characterization Zone.
c) A copy of a paper entitled "CANLEX Phase III full scale flow liquefaction test: instrumentation and construction" (Hofmann et al., 1996b)
CANLEX PHASE III
FULL SCALE LIQUEFACTION TEST: INSTRUMENTATION AND CONSTRUCTION

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ABSTRACT

Phase III of the Canadian Liquefaction Experiment (CANLEX) involved rapid loading of loose, saturated sand in an attempt to statically trigger a liquefaction flow slide. Extensive surface and subsurface instrumentation was installed to provide detailed information regarding the pore pressure and deformational behaviour of the loose sand foundation in response to the rapid loading. This paper describes the various types of instrumentation utilized, including electric piezometers, tilt sections, and surface survey monuments, as well as the data collection system utilized to handle large amounts of potentially rapidly changing data. The locations of the various types of instrumentation are illustrated. The piezometric and deformational response during different phases of construction are discussed.

RÉSUMÉ

La troisième phase de l'Expérience Canadienne de Liquéfaction (CANLEX) comportait le chargement rapide de sable lâche saturé dans le but de déclencher de façon statique un écoulement par liquéfaction. Un important réseau d'instrumentation a été installé en surface et dans le sous-sol pour obtenir des renseignements détaillés sur la pression interstitielle et sur la déformation de la fondation de sable lâche en réponse au chargement rapide. Cet article présente les différents instruments utilisés, incluant des piézomètres électriques, des nivelles, et des repères d'arpentage, ainsi qu'un système d'acquisition de données utilisé pour la manipulation d'une grande quantité d'entrées appelées à varier rapidement. La localisation des différents instruments est indiquée. La réponse piezométrique et le déformation au cours des différentes phases de la construction sont discutées.
INTRODUCTION

Phase III of the Canadian Liquefaction Experiment (CANLEX) involved attempting to statically trigger a liquefaction flow slide by rapidly loading a loose, saturated sand deposit. The rapid loading was carried out by pumping tailings into a reservoir bounded by a containment dyke constructed on a 10 m thick loose sand foundation deposit. The foundation sand was placed hydraulically into standing water in an abandoned borrow pit (J-pit), at Syncrude Canada Ltd., near Fort McMurray, Alberta, resulting in approximately 7 m of loose beach below water sand underlying 3 m of slightly more dense beach above water sand. The containment dyke was constructed as a clay-sand composite ring dyke overlying the hydraulically placed foundation sand. The test section of the embankment was constructed with clay shale to a height of 8 m, with sideslopes varying from 2:1 to 2.5:1. A 9 m high hydraulically placed sand berm, with sideslopes of 2.5:1, was tied into the completed clay embankment to form the back portion of the ring containment dyke. A Plan view of the test site in J-pit is shown in Figure 1.

Five main lines of instrumentation were installed in the loose foundation sands, as shown in Figure 1, prior to construction of the clay embankment to record the pore pressure and deformational response during construction and subsequent rapid loading. This paper describes the instrumentation and data collection systems installed at the CANLEX Phase III test site. The piezometric and deformational response of the loose sand foundation during drained construction of the clay embankment and rapid loading with tailings sand are included in the paper.

INSTRUMENTATION

Instrumentation utilized at the Phase III site can be divided into two classes: above and below ground systems. The above ground instrumentation system included a three dimensional video survey system utilized in conjunction with site surveys performed by Syncrude staff. The below ground instrumentation systems included electric piezometers, tilt sections, and settlement gauges.

Prior to installation of the above ground instrumentation, a topographic survey of the Phase III site was conducted. A similar survey was repeated once embankment construction was complete and again after the full scale liquefaction test.

In conjunction with “traditional” survey techniques, a three dimensional video survey system was installed to record the movements of the downstream slope and toe of the clay embankment during the rapid loading. The video survey system consisted of: three JVC TK-1280U video cameras mounted with a Rainbow MEA automatic iris electrically driven zoom lens, two video signal amplifiers, three Easy Reader II signal mixers, one Telcom Research TCG 550 time code generator, three JVC HR-S4900V SVHS video recorders, one 3-way video control switch, and one JVC TM-9U color video monitor.
The signal mixers, time code generator, three video recorders and video monitor were located in a modified trailer situated approximately 400 m east of the clay embankment. The left camera was situated approximately 220 m north of the trailer, the center camera was located approximately 5 m west of the trailer and the right camera was located approximately 180 m south of the trailer. Figure 1 shows the location of the video survey system with respect to the clay embankment.

The targets utilized by the video survey system included twenty-one day-time targets and twenty-one night-time targets. The day-time targets consisted of white painted soccer balls mounted on black, felt-covered, 1.2 m by 0.9 m by 9 mm thick plywood sheets attached to wooden posts. The night-time targets consisted of 0.3 m by 0.6 m white reflectors mounted on 330 mm x 300 mm PVC pipes. The targets were located on the downstream slope and toe of the embankment. The initial and post test positions of the targets were determined utilizing a Wild No. T2 directional transit. The positions of the targets were triangulated from four survey hubs located near each of the cameras with the fourth hub positioned southwest of the clay embankment to close the traverse (see Figure 1).

Piezometers

The first phase of the below ground instrumentation involved the installation of forty-two electric push-in piezometers in the loose sand foundation. The piezometers were installed with a cone penetration rig along four lines of instrumentation perpendicular to the long axis of the clay embankment. Figure 1 illustrates a plan view of the Instrumentation Lines 5, 1, 2, and 3, along which piezometers were installed. An elevation view of Piezometer Instrumentation Line 1 is shown in Figure 2.

The first thirty-six piezometers were installed at depths of approximately 5 m, 10 m, and 12 m by ConeTec Investigations Ltd., using their track mounted CPT unit. Two piezometers, PF19C2 and PF19T2, were installed in the loose sand foundation with a portable hand auger at a depth of 2 m and two foundation piezometers, PF1 and PF2, were installed in the native clay shale underlying the loose sand utilizing a drill rig. Once the clay embankment had been constructed to a height of 3 m, two piezometers, P99-A and P99-B, were installed by hand near the top of the three meter lift. At the end of construction of the 8 m high clay embankment, 41 of the 42 piezometers were functioning normally. Piezometer PF1, installed in the clay shale foundation, had not been working since the beginning of construction and it was not possible to repair it prior to the test.

Tilt Sections

The second phase of the below ground instrumentation was the installation of thirty-nine tilt sections. To provide information regarding the lateral deformation profile within the loose sand deposit during loading, five to six tilt sections were installed in each of seven boreholes, labeled SI-1 to SI-7. Instrumentation lines containing tilt sections were placed in between the piezometric instrumentation lines, with tilt sections located at the upstream toe, crest and
downstream toe of the embankment. An elevation view of one of the tilt section lines is shown in Figure 2, offset from Piezometer Line 1 towards Line 2 by 10 m. At the end of construction of the 8 m clay embankment, thirty-five of the thirty-nine tilt sections were operational.

The tilt sections incorporated an Accustar ratiometric electronic clinometer in a 2.1 m long by 100 mm diameter metal pipe. The input signal voltage was 10 volts with a 5 volt return signal when the clinometer was in the vertical position. The electronic clinometer was sealed in an air tight compartment at the bottom of the pipe enabling the remainder of the pipe to be filled with water prior to installation. The purpose of sealing the pipe full of water was to minimize sand intrusion into the pipe which could have potentially impeded movement of the signal wires during embankment construction and the event.

Each tilt section contained two chain links welded at opposite ends of the pipe section to facilitate correct alignment during installation and to attach a steel cable to each section. The purpose of the steel cable was to facilitate movement of the tilt sections as hinged members and to measure the change in length of the cable at the surface before and after the test as a secondary measure of lateral displacement. Prior to installation of a tilt line, a pin attached to a steel cable was driven into the clay shale. The cable was then threaded through the chain links of all tilt sections in the line. This pin acted as a hinged anchor at the base of the first tilt section, installed at the bottom of the borehole.

Settlement Gauges

The final below ground instrumentation that was installed were the settlement gauges. Four Sensotec low cost gauge pressure transducers were installed at depths of 5.5 m and 9 m adjacent to Instrumentation Lines 1 and 2, below the centerline and downstream toe of the clay embankment. Unfortunately, the settlement gauges malfunctioned and it was not possible to obtain any meaningful information from this instrumentation.

DATA ACQUISITION SYSTEM

The data acquisition system for the below ground instrumentation was located in a computer trailer situated approximately 20 m south of the clay embankment, as shown in Figure 1. Each instrument was connected to a 0.25 amp fuse to insure that the analog to digital (A/D) converter would not be damaged if a short circuit occurred.

The below ground instrumentation was separated into two groups and alternating rows of instrumentation were connected to one of two A/D converters, such that each converter contained an approximately equal number of piezometers, tilt sections, and settlement gauges. This was done to minimize the potential loss of data in the event of a system hardware failure. Each A/D converter was linked to an IBM compatible 486DX-66 personal computer (PC) equipped with National Instruments Labview software. The PCs, labeled Data Logger A
(DLA) and Data Logger B (DLB), were linked by RS-232 cables and LapLink software to a third 486DX-66 PC, Monitor A, equipped with a Colorado 700 tape drive to backup the data collected from DLA and DLB.

A fourth IBM compatible 486DX-66 PC, Monitor B, was installed to download data from Monitor A via a thinlan for on-site data processing and review. Monitor B was utilized to generate pore pressure plots with Excel v5.0 during the construction of the clay embankment to ensure that critical pore pressures that could trigger liquefaction were not prematurely attained. This fourth PC also proved to be useful for monitoring pore pressures during rapid infilling of the reservoir with tailings.

**INSTRUMENTATION RESPONSE AND PERFORMANCE**

**Piezometers**

During construction of the clay embankment, pore pressures were monitored on a continuous basis to ensure that critical pore pressures that could trigger liquefaction were not prematurely attained. The piezometers located along Instrumentation Lines 5 and 1, downstream of the embankment crest, showed the highest pore pressure response. Figure 3 shows a plan view of the zone in which highest pore pressures were recorded in the loose sand during embankment construction. In this region, most piezometers showed an increase in piezometric head during construction of between approximately 30 and 100% of the increase in thickness of the clay material placed. Cone Penetration Testing conducted prior to installation of piezometers at the test site, indicated that the sand appeared to be loosest in this region.

The maximum excess pore pressure head recorded during construction of the clay embankment was less than 8 m for all the piezometers installed in the loose foundation sand, except Piezometer P25A. Piezometer P25A, located below the crest of the embankment along Line 5, at a depth of 5 m below the top of the loose sand foundation, showed an increase in pore pressure head of approximately 8 m, in response to placement of the first 3 m thick lift of the clay embankment. However, as shown in Figure 4a, the excess pore pressures generated during construction of the clay embankment dissipated in less than 24 hours, resulting in only minor construction delays.

Piezometer PF2, installed between Lines 1 and 2 below the crest of the embankment in the clay shale foundation underlying the loose sand foundation, showed a high pore pressure response due to both the initial infilling of J-pit and the embankment construction, as shown in Figure 4b. The overall increase in piezometric head recorded in PF2 during construction was approximately 13 m. However, conventional limit equilibrium stability analyses conducted during placement of the first lift of clay, confirmed that the pore pressure response exhibited by PF2 did not threaten the short term stability of the clay embankment. Piezometer PF1, also installed in the clay shale underlying the loose sand, was not working properly during construction of the clay embankment.
Rapid infilling behind the clay embankment with tailings, took place from September 18 to September 20, 1995. The full water load was applied within 12 hours after infilling commenced and the full tailings sand load was applied within 36 hours. to a maximum height of 7 m near the location of the tailings sand outlet. All of the piezometers installed within the loose foundation sand were functional during this time. Piezometers P17A and P18A, located in the loose sand foundation, 5 m below the upstream toe of the clay embankment, showed the highest increases in piezometric head of 5.6 m and 6.7 m, respectively. The piezometers located beneath the upstream crest of the clay embankment, P25A&B, P13A&B, P14A&B and P15A,B&C, showed the next highest increase in pore pressure with an average rise in piezometric head of 3.5 m. The pore pressures along a given line, parallel to the long axis of the embankment, tended to be higher towards the south end, where Cone Penetration testing conducted prior to construction suggested that the foundation sand was loosest. All of the piezometers indicated relatively rapid dissipation of excess pore pressures once placement of the sand tailings was complete.

Figure 5 shows a plot of the change in piezometric head exhibited by Piezometer P18A, located along instrumentation Line 1 (see Figure 2), which showed the largest increase in piezometric head during rapid infilling. The plot of piezometric head elevation for P18A shows two distinct peaks. The first one is associated with the peak in water level behind the clay embankment and the second peak coincides with the peak level of the hydraulically placed tailings sand.

Tilt Sections

Profiles of the foundation movements caused by embankment construction were established based on tilt section data and indicated that the foundation sand generally deformed approximately 100 mm in a downstream direction. Movements of 100 mm near the ground surface, which gradually decreased to 0 mm at the base of the loose sand, were recorded in Boreholes SI-1, SI-4, SI-5 and SI-6. Due to malfunctioning tilt sections in Boreholes SI-2, SI-3 and SI-7, displacement profiles could not be constructed for these boreholes.

During rapid infilling behind the clay embankment with tailings, the deformations recorded by Tilt Sections SI-1, SI-4, SI-5 and SI-6 were within the resolution of the instrumentation. Therefore, it was concluded that movement of the loose sand foundation during rapid loading was minimal. This was later confirmed by the above ground survey.

CONCLUSIONS

A full scale liquefaction test was carried out at the Syncrude Canada site near Fort McMurray. Most of the instrumentation functioned well during the test and the data acquisition system that was in place successfully monitored the response of the loose sand foundation to the rapid infilling of the impoundment.
Liquefaction of the loose sand foundation did not occur during rapid loading; however, this may have been a result of geometric constraints. Geometry can play an important role in whether or not flow liquefaction is triggered that could result in a flow slide. The geometry and sequencing of the full scale test was limited by safety considerations in order to ensure maximum safety to personnel and equipment. It is possible that the geometry and sequencing of the test were not conducive to triggering a strain-softening response, even if the sand foundation itself was sufficiently loose to be strain-softening under the right loading conditions. On the upstream side of the clay embankment, the pore pressure response to the undrained loading appeared to correlate with the amount of additional total load that was rapidly placed on the sand foundation. Interpretation of the full scale liquefaction test is discussed in further detail by Robertson et al. (1996), in a companion paper written for this conference.

ACKNOWLEDGMENTS

This work was partly supported by CANLEX (Canadian Liquefaction Experiment), a project funded through a Collaborative Research and Development Grant from the Natural Sciences and Engineering Research Council of Canada (NSERC), Syncrude Canada Ltd., Suncor Inc., Highland Valley Copper, B.C. Hydro, Hydro Quebec and Kennecott Corporation. The collaboration also includes the engineering consulting companies, EBA Engineering Consultants Ltd., Klohn-Crippen Consultants Ltd. AGRA Earth and Environmental Ltd., Golder Associates Ltd., and Thurber Engineering Ltd., as well as, faculty, staff and students from the Universities of Alberta, British Columbia, Laval and Carleton.

Figure 1: Phase III Test Site Instrumentation Location Plan
Figure 2: Piezometer and Tilt Section Instrumentation Cross-Section

Figure 3: Plan View of Instrumentation Lines Through Embankment Showing Zone of Highest Construction Pore Pressures
Figure 4a: Piezometric Head Exhibited by P25A during Embankment Construction

Figure 4b: Piezometric Head Exhibited by PF2 during Embankment Construction

Figure 5: Piezometric Head Exhibited by P18A during Rapid Loading
d) A copy of a paper entitled “Analysis of CANLEX liquefaction embankments: prototype and centrifuge models” (Puebla et al., 1996)
ANALYSIS OF CANLEX LIQUEFACTION EMBANKMENTS:
PROTOTYPE AND CENTRIFUGE MODELS

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ABSTRACT

A major aim of the Canadian Liquefaction Experiment was to verify our ability to predict liquefaction phenomena. Towards this purpose, a field event comprising a storage impoundment was constructed at the Syncrude site at Fort McMurray, Alberta, and loaded in an attempt to induce a static liquefaction failure. Sophisticated stress deformation analyses were carried out to predict the response of the impoundment structure and related centrifuge model tests. Drainage was a very important factor during the loading and was directly incorporated in the analysis procedure. The predicted results are in good agreement with the measurements.

RÉSUMÉ

Un des objectifs principaux de l’Expérience Canadienne sur la Liquefaction était de vérifier notre capacité de prédire le phénomène de la liquéfaction. Dans la poursuite de cet objectif, un essai sur le terrain comprenant un parc de résidus fut construit sur le site Syncrude à Fort McMurray, en Alberta. Un chargement fut appliqué dans l’espoir de créer une rupture de liquéfaction statique. Des analyses sophistiquées des contraintes et déformations furent entreprises dans le but de prédire la réponse du système et celle d’un modèle réduit en centrifugeuse. Le drainage était un aspect important durant le chargement et fut incorporé directement à la procédure d’analyse. Une bonne corrélation fut établie entre les résultats ainsi prédits et les mesures expérimentales.
INTRODUCTION

The Canadian Liquefaction Experiment (CANLEX) involves three major aspects: the characterization of sand in order to predict its liquefaction response, a full scale liquefaction event at the Syncrude site near Fort McMurray, Alberta, and the calibration of numerical models to predict this phenomenon. To help in the design of the field event, centrifuge model tests with numerical simulations were also carried out.

The numerical stress-strain model was calibrated by comparison with laboratory element tests and the analysis procedure verified by comparison with centrifuge tests. The calibrated numerical model was then used to analyze the proposed field event. The response predicted by the model in terms of deformation and porewater pressure is presented in this paper.

PROPOSED LIQUEFACTION EVENT

The purpose of a full scale event was to statically trigger a liquefaction flow slide by rapidly loading a loose saturated sand deposit. The event is described in detail by Byrne et al. (1995b) and involved the construction of a test embankment over the loose target layer as shown in Figure 1.

The foundation sand was placed hydraulically into standing water in an abandoned pit, J-Pit, up to Elevation 318m. A level platform was then formed up to Elevation 321m by placing tailings sand above the water. A clay dyke 8m high with sideslopes of 2.5:1 (hor:vert) was constructed over the tailings. A compacted sand containment structure 10m high was then constructed over predominantly natural firm ground. Rapid loading was brought about by pumping tailing behind the clay dyke. The site was characterized and instrumented as discussed by Byrne et al. (1995b).

ELASTIC-PLASTIC STRESS-STRAIN MODEL AND ANALYSIS PROCEDURE

The stress-strain model has been presented by Byrne et al. (1995a). The sand skeleton is modelled as strain hardening elastic-plastic in shear. The yield loci are lines of constant stress ratio or friction angle. Yield with strain hardening occurs as the stress ratio or developed friction angle increases leading to plastic shear strain. The stress ratio / plastic shear strain relationship is assumed to be hyperbolic, and the slope of this curve gives the plastic shear modulus. Plastic volume changes are based on a non-associated flow rule.

The yield function $f$ is defined by:

$$f = \sigma'_1 - \sigma'_3 N_0$$

where:

$\sigma'_1$ = major principal stress,
$\sigma'_3$ = minor principal stress,
$N_0 = (1 + \sin \phi_u) / (1 - \sin \phi_u),$
$\phi_u$ = developed friction angle.
The plastic potential function that corresponds to a non-associated flow rule can be written as:

\[ g = \sigma'_{1} - \sigma'_{3} N_{\psi} \]  

where: \( N_{\psi} = (1 + \sin\psi) / (1 - \sin\psi) \), and \( \psi \) is similar to a dilation angle, and is related to the constant volume friction angle, \( \phi_{cv} \) and \( \phi_{d} \) by:

\[ \sin\psi = (\sin\phi_{cv} - \sin\phi_{d}) \]  

The model captures the drained or skeleton behaviour of sand as shown schematically in Figure 2. The undrained behaviour is captured by imposing the volumetric constraint caused by the fluid stiffness.

The analyses were carried out using the computer code FLAC (Fast Lagrangian Analysis of Continua) version 3.3 (Cundall, 1995) with the stress-strain model described above. This program uses a finite difference method and establishes dynamic equilibrium using a step by step explicit time domain procedure. Large displacements and strains are approximated by updating the nodal coordinates of the grid.

**NUMERICAL SIMULATION OF LABORATORY TESTS**

Using the stress-strain model and the FLAC code described above, the undrained response observed in triaxial compression and extension tests, as well as simple shear, was captured. A comparison between the observed and the predicted response for a confining stress of 100 kPa is shown in Figure 3. The model parameters were obtained by back analysis of the laboratory curves and the same parameters were used over a range of confining stresses.

As it can be seen in Figure 3 the sand response is strain-hardening in triaxial compression, while in triaxial extension, as well as in simple shear, it is strain-softening. The response is different due to the direction of loading and it is essential for the numerical model to capture this. This is achieved by making the plastic shear modulus a function of the direction of the major principal compressive stress, \( \sigma'_{1} \), that is an anisotropic plastic response.

**CENTRIFUGE MODELLING**

The centrifuge test modelling has been described and analyzed by Byrne et al. (1995c). Test 1 Activity II-6D (Phillips and Byrne, 1994) was chosen for detailed description and analysis because liquefaction phenomena were observed and it resembles the proposed field event. A profile of this test is shown in Figure 4. Herein, only Table 1 comparing the measured and predicted pore pressures and displacements will be presented.
TABLE 1. Comparison of results - Centrifuge Test II-1 (After Byrne et al. 1995c)

<table>
<thead>
<tr>
<th>INSTRUMENT</th>
<th>MEASURED</th>
<th>PREDICTED</th>
</tr>
</thead>
<tbody>
<tr>
<td>PPTA</td>
<td>52 kPa</td>
<td>45 kPa</td>
</tr>
<tr>
<td>PPTB</td>
<td>80 kPa</td>
<td>80 kPa</td>
</tr>
<tr>
<td>PPTC</td>
<td>68 kPa</td>
<td>52 kPa</td>
</tr>
<tr>
<td>LDT1</td>
<td>(-0.34 m)</td>
<td>(-0.39 m)</td>
</tr>
<tr>
<td>LDT2</td>
<td>&gt;0.14 m</td>
<td>0.34 m</td>
</tr>
</tbody>
</table>

From Table 1 it can be seen that measurements and predictions are in good agreement.

ANALYSIS OF FIELD EVENT

The model parameters used in the analysis of the field event were obtained from fitting the laboratory data from triaxial compression and extension, as well as simple shear tests. The parameters for the rest of the materials were based on Duncan et al. (1980) and Byrne et al. (1987). To simulate the behaviour of the contained sand, it was assumed that this slurry behaved as a heavy fluid.

The porewater pressure, in terms of contours, and the displacement pattern obtained by numerical simulation under undrained conditions are presented in Fig. 5. Results of the coupled stress-flow analysis are shown in Figure 6. This was simulated by allowing seepage from the water boundary overlaying the contained sand.

The maximum predicted displacement occurred in the toe area and was about 10cm for the undrained condition and 5cm for the coupled stress-flow condition.

A comparison between the measured and predicted values of excess porewater pressure at the locations shown in Figure 7, as well as displacements at the toe of the embankment, are shown in Table 2, for the undrained and coupled stress-flow analyses. The predicted values and the field observations compare very well.

TABLE 2. Measured and predicted values of excess porewater pressure and displacement.

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>MEASURED</th>
<th>PREDICTED</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>COUPLED STRESS-FLOW</td>
</tr>
<tr>
<td>P25-A</td>
<td>PORE</td>
<td>38.3</td>
</tr>
<tr>
<td>P24-A</td>
<td>PRESSURE</td>
<td>14.7</td>
</tr>
<tr>
<td>P26-A</td>
<td>[kPa]</td>
<td>9.3</td>
</tr>
<tr>
<td>TOE OF DYKE</td>
<td>DISPL [cm]</td>
<td>5</td>
</tr>
</tbody>
</table>

Figure 8 shows contours of porewater pressure and the displacement pattern for a hypothetical case in which the embankment would have been 16m high instead of 8m. Displacements in excess of 2.5m occur for this case.
CONCLUSIONS

The results of the analysis show: 1) The predicted displacements and porewater pressures were in reasonable agreement with those measured in the field event. 2) The reason for small displacements beyond the toe was the lack of excess porewater pressure in this region and 3) Analysis show that had the embankment been 16m high instead of 8m, then rapid undrained loading would have triggered static liquefaction.

ACKNOWLEDGMENTS

This research was partially funded by grants from NSERC and CANLEX Project. This Project involves collaboration between industrial partners, including BC Hydro, Quebec Hydro, Syncrude Canada Ltd. and Suncor, geotechnical consultants and the Universities of Alberta, British Columbia. Laval, Carleton and Sherbrooke. The authors wish to thank Miss C. Frenette for her French translation.

REFERENCES


FIGURE 1. Field event geometry.

a. Shear behaviour

b. Shear Volume coupling

c. Volumetric behaviour
d. Contractive and Dilative States

FIGURE 2. Schematic drained behaviour of sand.
a. Undrained response of element triaxial tests

b. Undrained response of element simple shear test

FIGURE 3. Undrained response of element tests.

FIGURE 4. Centrifuge test geometry - stage II-1.

FIGURE 5. Embankment response to loading under undrained conditions.
FIGURE 6. Embankment response to loading under coupled stress-flow conditions.

FIGURE 7. Location of piezometers in the field.

FIGURE 8. 16m high embankment response to loading under undrained conditions.
e) A copy of a paper entitled "Numerical analysis of the CANLEX Phase III field event" (Soroush et al., 1996)
Numerical Analysis of the CANLEX Phase III Field Event

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ABSTRACT

Phase III of the CANLEX experiment involves constructing a 10 m high embankment over tailings sands placed under water. The failure of the embankment was to be triggered by rapid loading behind the embankment using hydraulically placed sand. The construction and the behavior of the embankment was modeled using a finite element model that incorporates the collapse plasticity constitutive relationship. The model has been tested and calibrated using centrifuge experiments conducted for the CANLEX project.

The clay embankment was modeled in stages under undrained conditions. The sand below was allowed to consolidate under the weight of the embankment. Placement of the sand behind the embankment was simulated in layers to examine the effect of different rates of loading. Various S₀/p’ ratios for the sand below water were used to explore the effect of the undrained strength of the material on the potential of failure and the failure mechanism. Depending on the S₀/p’ ratio, the embankment may or may not fail under the loading imposed by the sand behind the embankment. Low rate of loading results in dissipation of pore pressure and leads to higher undrained shear strength. Steady state seepage under the embankment was also considered to examine the effect of seepage on the static liquefaction potential of the sand.

RÉSUMÉ

La Phase III de CANLEX comporte la construction d’un remblai d’une hauteur de 10 m recouvrant des résidus de mine sableux submergés. La rupture du remblai devait être déclenchée par un chargement rapide en utilisant du sable placé hydrauliquement derrière le remblai. La construction et le comportement du remblai ont été modélisés par éléments finis en tenant compte de la loi constitutive à la rupture en plasticité. Ce modèle a été testé et établi avec des essais en centrifugeuse effectués dans le cadre de CANLEX.
Le remblai d’argile a été modélisé par étapes en conditions non drainées. On a permis au sable sous-jacent de consolider sous le poids du remblai. La mise en place de sable derrière le remblai a été simulée par couches successives afin d’étudier l’effet de différentes vitesses de chargement. Différents rapports $S_u/p’$ pour le sable submergé ont été utilisés dans le but de clarifier l’influence de la résistance non drainée du matériau sur le potentiel de rupture et sur le mécanisme par lequel cette rupture se produit. Selon la valeur du rapport $S_u/p’$, on a trouvé que le remblai simulé pouvait céder ou non sous l’effet de la charge imposée par le sable. Une vitesse de chargement faible permet la dissipation de la pression interstitielle et s’accompagne d’une augmentation de la résistance au cisaillement en conditions non drainées. L’écoulement en régime permanent sous le remblai a aussi été considéré pour évaluer l’influence de l’écoulement sur le potentiel de liquéfaction statique du sable.

INTRODUCTION

As a part of Phase III of the CANLEX experiment, a full-scale liquefaction event was designed and carried out at the Syncrude site near Fort McMurray, Alberta, during the Summer of 1995. Failure of an embankment constructed over under-water placed Syncrude tailings sands was to be triggered by rapid loading behind the embankment. The event was proposed in a manner to provide a static liquefaction failure in the field similar to the upstream liquefaction failures occurred at the Suncor Tar Island Tailings Pond in the 1970’s. The design of the event was based on field experience, centrifuge testing and numerical study. The numerical study was carried out to understand the liquefaction process of sand, to analyze and make predictions of both the centrifuge tests and the full-scale event, to compare the numerical forecast with the field measurements, and to evaluate the current capabilities of the numerical modeling.

A numerical model based on the concept of steady state and collapse surface was employed to analyze the field event. The model is the same as the one that successfully explained flow liquefaction failures in the centrifuge model tests (Chan et al., 1995). The results of the numerical analysis of the centrifuge experiments were promising which demonstrated the ability of the model in simulating the behavior of liquefiable materials. The computer model is based on the finite element method with an elasto-plastic constitutive relationship which can simulate the undrained behavior of sands. When loose saturated sands are sheared under undrained conditions, the generation of pore water pressures due to contraction of the soils results in a substantial loss in strength. Sladen et al. (1985) introduced a collapse surface which defines the maximum shear stress a sand can sustain before it strain softens to the steady state conditions. The collapse surface, therefore, defines the state which triggers strain softening for loose sands subjected to undrained loading.

The elasto plastic model introduced by Gu et al. (1994) is adopted in this analysis. Liquefaction is considered to be an undrained process in the model. The model defines 3 zones of soil behavior as shown in Figure 1. In zone 1, elastic or strain hardening behavior occurs during loading by shear stress. In zone 2, strain softening behavior occurs. The strain softening behavior in this zone is simulated by a hyperbolic strain softening model. In zone 3, elastic or strain hardening behavior may occur which results in decrease of excess pore pressures. The numerical model is based on an effective stress analysis which calculates excess pore water pressures generated during undrained deformation.
FIELD EVENT

Plews (1994) proposed a preliminary field event in which triggering of flow failure in a 10 m under-water placed tailings was intended directly by rapid construction of an embankment over the tailings. The maximum height of the embankment was initially proposed to be 15 m. This preliminary design of the event was numerically analyzed (Soroush and Chan, 1995) and the results were submitted to the CANLEX. The results of the analysis helped in designing the required instrumentation for the event. On the basis of a number of influential factors and the analysis results, the final design of the event was changed. Major considerations were safety to workers, safety to equipment, cost, and construction procedure compatible with Syncrude’s operations.

In the final design of the event, the maximum height of the clay embankment was limited to 10 m (8 plus 2) based on safety concerns and reduced to 8m due to cost restrictions. The failure of the embankment and the tailings sands were to be triggered by rapid loading behind the embankment using hydraulically placed tailings. The trigger mechanism was agreed to be a static one comprising both a rapid loading as well as boundary water loading, leading to transient seepage. Details of the liquefaction event planning were given by Byrne et al. (1995). The event was designed to take place in J-pit in Syncrude Canada site near Fort McMurray. The pit is approximately 11 m deep, and at the time of construction was almost full of water. Figure 2 illustrates a cross section of the site. The information on site characterization indicated that about 1.5 m of soft clay exists between the tailings and the foundation.

NUMERICAL MODELING

The field event was modeled using the finite element computer program PISA™. The program has incorporated the collapse surface plasticity model for simulating undrained behavior of sands. A plane strain finite element mesh was used to model the event. A certain depth of the foundation was included in the finite element mesh and the mesh was extended laterally far enough to minimize boundary effects. Initial in-situ stresses in the tailings sands were generated using the switch-on-gravity technique. The material parameters used in the analysis are shown in Table 1. These parameters were determined based on field and laboratory tests. Strength and deformation parameters of the tailings were estimated from the average cone resistance obtained during site characterization. Based on the information on site characterization, the unit weight and the strength parameters of the top 3 m of the tailings were assumed to be slightly higher. Pore pressure parameters used in the undrained analysis are the same values used in the analyses of the centrifuge tests (Chan et al., 1995).

Modeling of Embankment Construction

The construction of the clay embankment over the submerged tailings was simulated in 3 ways representing drained, undrained and partially drained conditions. A drained analysis was performed to simulate slow construction of the embankment. In this analysis, the tailings sands were considered to be fully drained during the embankment construction. This may not represent the real condition due to the fact that a finite amount of time is required for pore pressures to dissipate. However, the drained condition represents one extreme case in the evaluation of the
stability of the embankment. The finite element results of this analysis indicated that no significant deformations occurred during the embankment construction.

An undrained effective stress analysis was carried out to model the rapid construction of the embankment over the tailings with no dissipation of excess pore water pressures in all construction lifts. This represents the other extreme condition which may be experienced by the tailings sands. Results of the analysis showed that failure will occur at the early stage of the embankment construction. Failure was indicated by non convergence of the numerical solutions.

The above two cases represent extreme conditions the sand may experience during the embankment construction. Realistically, partial drainage (or complete drainage, depending on the permeability of the tailings sand and the rate of construction) will occur. In order to simulate partial dissipation of pore pressures between each lift of fill placement, an undrained analysis was carried out with excess pore pressures in the sand being reduced between lifts. The pore pressure was reduced to sufficiently low values which would allow a stable construction of the embankment.

Results of the partially drained analysis are plotted in Figure 3, which represents the maximum allowable excess pore water pressure at the mid-depth of the tailings sands (under the center line of the embankment) in order to avoid instability during the embankment construction. This figure was provided to Syncrude as a guide for construction control during the embankment construction. During construction, responses of all piezometers located in the tailing sands, except two piezometers in line 5, did not exceed the recommended values in Figure 3. This resulted in a safe construction of the clay embankment.

<table>
<thead>
<tr>
<th>Material (depth)</th>
<th>( \gamma' ) (kN/m³)</th>
<th>( E ) (kN/m²)</th>
<th>( \nu )</th>
<th>( \phi_s )</th>
<th>( \phi_c )</th>
<th>( A_0 )</th>
<th>( A_m )</th>
<th>( S_u/p' )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tailings (low 8 m)</td>
<td>8.95</td>
<td>14,000</td>
<td>0.49</td>
<td>34</td>
<td>24</td>
<td>0</td>
<td>5</td>
<td>0.2, 0.15, 0.1</td>
</tr>
<tr>
<td>Tailings (top 3 m)</td>
<td>9.4</td>
<td>18,000</td>
<td>0.49</td>
<td>36</td>
<td>25</td>
<td>0</td>
<td>5</td>
<td>0.2, 0.15, 0.1</td>
</tr>
</tbody>
</table>

Key: \( \phi_s \) = friction angle at steady state;  
\( \phi_c \) = angle of collapse surface;  
\( A_0, A_m \) = pore pressure A parameters in liquefaction analysis;  
\( S_u/p' \) = undrained steady state strength ratio.

**Modeling of Loading due to Contained Tailings**

A number of analyses were carried out to investigate the effect of contained tailings on the stability of the embankment. In these analyses, contained tailings were assumed to be placed after all excess pore water pressures in the tailings sands due to the embankment construction had been dissipated. This assumption was made based on the observed piezometers response which indicated relatively fast dissipation of the excess pore water pressure in the tailings sands. The analyses are categorized into three main cases:
Case 1- The contained tailings were assumed to be piped rapidly between the embankment and the compacted sand dyke so that the tailings imposed undrained loadings on the structure. No dissipation of excess pore water pressures was assumed during filling.

Case 2- Steady state seepage in the tailings sands was assumed during filling of the contained tailings. In this case, the tailings sands and the embankment were loaded under drained conditions.

Case 3- The contained tailings were assumed to be placed in 3 lifts with reduction of excess pore pressures in the tailings sands between lifts. This analysis was performed to study the effect of the loading rate on the stability of the embankment.

In situ testing, including CPT and SPT, before the embankment construction indicated that the tailings sands are generally loose although highly non-homogeneous. Based on the in situ test results, the average undrained steady state strength ratio of the tailings sands ($S_u/p'$) before the embankment construction was estimated to be about 0.1. $S_u/p'$ ratios in the range of 0.1-0.2 were used in the analysis to examine its sensitivity on the results. In order to investigate the effect of higher strength at the top 3 m of the tailings sands on the possibility of a liquefaction flow failure, analyses were carried out with higher $S_u/p'$ ratios for the top 3 m of the tailings. An increase in the $S_u/p'$ ratio of the tailings sands under the clay embankment was also expected due to consolidation. In order to examine the effects of this increase in $S_u/p'$ ratio on the results, a number of analyses were carried out with higher $S_u/p'$ ratios for the tailings under the embankment. Cases 1, 2 and 3 analyses with different $S_u/p'$ ratios are summarized in Table 2.

Results of Analyses

Based on the information summarized in Table 2, the major findings of the liquefaction analysis are:

1) Rapid filling of the contained tailings behind the clay embankment, with no dissipation of excess pore water pressures during filling, will trigger liquefaction flow failure in the tailings sands if the undrained steady state strength ratio of the tailings sands does not exceed 0.1.

2) Liquefaction flow failure (if any) will occur after the undrained loading before the steady state seepage is established in the tailings sands.

3) Results of case 3 analysis indicated that if loading is not fast enough, liquefaction failure will not occur. Rapid loading of the contained tailings is essential in order to trigger a flow failure. It was therefore recommended that the contained tailings should be filled continuously and be completed in about 24 hours to minimize dissipation of excess pore water pressures in the tailings sands. Figure 4 shows contours of computed excess pore water pressures at the end of loading for case 3 analysis.

4) If the $S_u/p'$ ratio of the tailings sands under the embankment is increased to 0.2 due to construction of the clay embankment, failure will not occur.
| TABLE 2. Summary of the analyses of loading due to contained tailings. |
|---|---|---|---|
| Constant $S_u/p'$ | Case 1 | Case 2 | Case 3 |
| 0.1 | Failure | No Failure | No Failure |
| $>0.1$ | No Failure | No Failure | No Failure |
| $(S_u/p')$ of the top 3 m tailings is higher* | 0.15 | Failure | No Failure |
| $>0.2$ | No Failure | No Failure | No Failure |
| $S_u/p'$ of tailings right under embankment is higher* | 0.15 | Failure | No Failure |
| 0.2 | No Failure | No Failure | No Failure |

* A value of 0.1 is used for $S_u/p'$ ratio of the low 8 m of the tailings.
* A value of 0.1 is used for $S_u/p'$ ratio of the tailings which are not located under the clay embankment.

**POST-EVENT COMMENTS**

The field event began on September 18, 1995. Water and tailings were poured in the cell behind the clay embankment. The water level was raised to a height of 7.5 m and the sand was placed to a height of 7 m. Loading lasted for about 36 hours and the embankment moved very little during the loading. A number of possible factors may have contributed to the stability (non-failure) of the embankment. Major factors are listed as follows:

1. **Rate of Loading Compare to Rate of Excess Pore Pressure Dissipation**

   Although from a feasibility point of view the loading rate was relatively fast, it was not fast enough to induce high enough excess pore pressures in the tailings to reach a state of instability. It is important that the rate of loading be studied together with the rate of dissipation of excess pore water pressure. Piezometer measurements during the clay embankment construction showed that the tailings sands are highly permeable. Almost half of the induced pore water pressures were dissipated after 12 hours from the beginning of lift construction. Also piezometer response during loading behind the embankment indicated that a large amount of excess pore pressures had been dissipated during loading. Results of case 3 analysis showed that if the loading was not fast enough, i.e. most of the excess pore pressures dissipate during the loading, no liquefaction flow failure will occur. Table 3 compares the observed and calculated changes in water level for case 3 in Line 1 of the instrumented section. The piezometers location in Line 1 is shown in Figure 5. Line 1 instrumented section showed the highest pore pressure response during the experiment.

   Figure 6 compares the measured excess pore water pressure at piezometer P18-A during loading with calculated excess pore water pressures for cases 1, 2 and 3. As indicated, the computed excess pore water pressure from case 1, which assumes fully undrained conditions, is much higher than the observed excess pore water pressure. However, the excess pore water pressures calculated from cases 2 and 3 are close to the measured value. These results indicate that the assumption of fully undrained loading was not realistic due to relatively high permeability of the tailings sands.
TABLE 3. Comparison between the observed and calculated changes in water level for case 3 in Line 1 of the instrumented section (change in water level in meter).

<table>
<thead>
<tr>
<th></th>
<th>P18A</th>
<th>P13A</th>
<th>P13B</th>
<th>PF19C2</th>
<th>P09A</th>
<th>PF19T2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Observed</td>
<td>6.8</td>
<td>3.4</td>
<td>3.8</td>
<td>1.8</td>
<td>1.6</td>
<td>0.7</td>
</tr>
<tr>
<td>Calculated</td>
<td>7.1</td>
<td>2.9</td>
<td>3.4</td>
<td>1.2</td>
<td>1.0</td>
<td>0.9</td>
</tr>
</tbody>
</table>

2. Stress Path Dependency of the Undrained Behavior of Tailings Sands

Recent laboratory testing carried out after the field event on both undisturbed and reconstituted samples of the tailings sands showed that the potential for liquefaction of Syncrude sand at a given void ratio is dependent on the stress path during undrained shear as well as the initial consolidation stress. The test results showed that for a given initial state (void ratio and confining stress) the sand under undrained loading is slightly strain softening in triaxial compression loading and highly strain softening in triaxial extension loading. The test results in simple shear condition tend to lie between that of the triaxial compression and extension tests. The undrained steady state strength ratio based on undisturbed samples was about 0.3 in triaxial compression and about 0.03 in triaxial extension.

Based on the above results, it is useful to investigate stress paths in the tailings sands during loading. Figure 7 shows contours of the angle \((\alpha)\) between the major principle stress \((\sigma_1)\) and the depositional (vertical) direction. As indicated most of the tailings, including the tailings under upstream of the embankment are sheared predominantly in compression. Only the surface material near the toe are subjected to extension mode. Therefore it is reasonable to conclude that a rigorous liquefaction analysis based on the above undrained steady state strength ratios (0.3 for compression and 0.03 for extension) most likely will result in standing (non-failure) of the embankment.

3. Relative Density of the Tailings Sands

The field and laboratory tests data showed that the tailings are strongly non-homogeneous. The laboratory tests on the undisturbed frozen samples of the sand indicated that most of the samples lied on the dense side of the steady state line; hence, most of them may not behaved in a strain softening manner during loading. Despite both CPT and SPT results indicated that the tailings sands are generally loose, they may not be loose enough, compared to the steady state line, to have static flow liquefaction.

In addition to the above factors, the method by which the tailings sands were placed could have some effects on its response. In the field experiment the tailings sands were deposited under water. Laboratory tests on Syncrude sands showed that the sands reconstituted by air pluviation is much more prone to liquefaction than its water pluviated counterpart (Vaid et al., 1995). It should be noted that the sand in the centrifuge tests was deposited by air pluviation.

Summary

This paper described numerical analyses of the field event using an elasto plastic model. Results of the partially drained analysis of the clay embankment construction helped the embankment to be constructed safely in the shortest possible time. The observed pore pressures during the
embankment construction were comparable with the computed values, which were recommended for safe construction of the embankment.

The effect of loading of the contained tailings was numerically simulated under drained, undrained and partially drained conditions. The model predicted that rapid loading of the underwater deposited tailings sands can trigger liquefaction flow failure provided that (1) the undrained steady state strength ratio of the tailings does not exceed 0.1 and (2) the loading be rapid with no dissipation of excess pore pressures. The embankment did not fail and moved very little during the loading. A number of possible influential factors, including the relatively high permeability of the tailings and the stress-path dependency of the undrained behavior of the tailings sands were discussed.

ACKNOWLEDGMENTS

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REFERENCES


FIGURE 1. Undrained model for liquefiable soils.

key: q = deviatoric stress
p' = mean normal effective stress
Kp = peak strength
Kr = residual strength
u = pore pressure
u_p = equivalent plastic strain
\( \alpha_c \) = angle of collapse surface
\( \alpha_H \) = angle of Hvorslev surface

FIGURE 2. Cross-section of the field event.

Note: negative sign means compressive (positive pore pressure).

FIGURE 3. Maximum excess pore pressure at mid-depth of tailings sands (under centre-line of embankment) for safe construction of embankment.

FIGURE 4. Contours of excess pore pressures at the end of loading in case 3 analysis (kPa).
FIGURE 5. Location of piezometer tips of Line 1 instrumented section.

FIGURE 6. Comparison of observed and computed pore pressures at P18A, Line 1.

FIGURE 7. Contours of the angle (α) between major principle stress (σ₁) and depositional (vertical) direction (α is in degree).
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