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CANLEX (Canadian Liquefaction Experiment): A One Year Update

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SYNOPSIS The Canadian Geotechnical engineering community has embarked on a major study regarding the liquefaction of sand entitled The Canadian Liquefaction Experiment (CANLEX) through a collaborative effort of industry, engineering consultants and university participants, with the support of the Natural Sciences and Engineering Research Council of Canada (NSERC). The study is examining the characterization of sand for dynamic and static liquefaction. The project was started in 1993 and is expected to last at least 3 years with equal funding by both industry and NSERC for a total of about C$1.8M. This paper provides a brief progress report on the Project. Three test sites have been selected and characterized using in-situ testing, conventional sampling as well as in-situ freezing to obtain undisturbed samples. Laboratory testing is underway on both reconstituted samples and undisturbed samples. A full scale liquefaction event is planned for year three of the Project and a feasibility study regarding the event has been completed. As part of the planning for the liquefaction event some preliminary centrifuge testing has been carried out. A static liquefaction flow failure has been successfully produced in the centrifuge. As part of the Project, a set of definitions for liquefaction have been defined and a flow chart developed to aid in the liquefaction analyses.

INTRODUCTION

The Canadian Geotechnical engineering community has embarked on a major study regarding the liquefaction of sand entitled The Canadian Liquefaction Experiment (CANLEX) through a collaborative effort of industry, engineering consultants and university participants, with the support of the Natural Sciences and Engineering Research Council of Canada (NSERC). The study is examining the characterization of sand for dynamic and static liquefaction. The project was started in 1993 and is expected to last at least 3 years, with equal funding by both industry and NSERC for a total of about C$1.8M. The industry participants are B.C. Hydro, Quebec Hydro, Syncrude Canada Ltd., and Suncor Inc. The geotechnical consultants are AGRA Earth and Environmental Limited, EBA Engineering Consultants Ltd., Golder Associates Ltd., Klohn-Crippen Consultants Ltd.and Thurber Engineering. The study is also collaborative with faculty and students from the University of Alberta, University of British Columbia, Carleton University, Universite de Laval and the Universite de Sherbrooke.

In many parts of the world large structures are constructed on or comprised of sand. Examples of such structures are tailings impoundments developed by the mining industry and some of the major earth dams used for hydro electricity generation. The behavior of loose sand deposits can be difficult to predict but can have a significant financial impact on these types of structures. The characterization of loose sand deposits is an area of uncertainty in geotechnical engineering. Unlike clay deposits, it is almost impossible to obtain undisturbed samples of loose saturated sand, especially at depth, using conventional methods. The CANLEX Project enables a focused, coordinated effort by Canadian geotechnical engineers to advance our understanding in the areas of site characterization and soil liquefaction. The main objectives of the project are;

- Develop test sites to study sand characterization
- Develop economical undisturbed sampling techniques, considering that in-situ freezing is seen as the most promising technique
- Obtain a greater understanding of soil liquefaction

The Project has been divided into Phases, with each Phase extending for approximately one year. This paper predominately describes the progress during Phase I.

LIQUEFACTION

The CANLEX Project has agreed on terminology to be used to define soil liquefaction (Robertson, 1994). The following section briefly describes this terminology.

If a soil structure, such as an earth dam or tailings dam, is composed entirely of a strain softening soil and the in-situ gravitational shear stresses are larger than the ultimate steady state strength (i.e. a relatively steep slope), a catastrophic collapse and flow slide can occur if the soil is triggered to strain soften. The collapse can be triggered by either cyclic or monotonic undrained loading. If a soil structure is composed entirely of a limited strain softening soil and the in-situ gravitational shear stresses are larger than the quasi-state strength, a catastrophic flow slide is unlikely. However, large deformations can occur before the soil stiffens as it strain hardens towards its ultimate state.

If a soil structure is composed entirely of strain hardening soil, undrained collapse and a flow slide can not occur and deformations will, in general, be small. If a soil structure is composed partly of strain softening (SS) and strain hardening (SH) soil and the SS soil is triggered to strain soften, a collapse and flow slide will only occur if, after stress redistribution due to the softening of the SS soil, the SH soil can not support the gravitational shear stresses. The trigger mechanism can be cyclic, such as earthquake loading, or monotonic, such as a rise in the ground water level or rapid undrained loading. Gu et al. (1993a) used the collapse surface approach to explain the failure of the Lower San Fernando Dam shortly after the 1971 San Fernando earthquake. Gu et al. (1993b) also used the collapse surface approach to explain the continued buildup of pore pressure and deformation after the 1987 Superstition Hills earthquake at the Wildlife Site in the Imperial Valley, California.

During cyclic undrained loading, almost all granular soils develop positive pore pressures due to the contractant response of the soil at small strains. If there is shear stress reversal, the effective stress state can progress to the point of zero effective stress. For shear stress reversal to occur, ground conditions are generally level or gently sloping. When a soil element reaches the condition of zero effective stress, the soil has very little stiffness and large deformations can occur during cyclic loading. For very dense soils, the cyclic loading may not be sufficient to reduce the state to zero effective stress and hence, deformations essentially stop, except those due to local pore pressure redistribution. Gu et al. (1993b) showed that the deformations due to pore pressure redistribution were very small at the Wildlife Site in the Imperial Valley. If there is no shear stress reversal, the stress state can not reach zero effective stress and cyclic mobility with limited deformations will occur.

PROPOSED DEFINITIONS OF LIQUEFACTION

Based on the above description of soil behavior in undrained shear, the following definitions of liquefaction are suggested.

Flow Liquefaction

- Requires strain softening response in undrained loading resulting in constant shear stress and effective stress, (i.e. ultimate steady or critical state).
- Requires that in-situ shear stress is greater than undrained residual or steady state shear strength.
- Flow liquefaction can be triggered by either monotonic or cyclic loading.
- For failure of a soil structure to occur, such as a dam or a slope, a sufficient volume of material must show strain softening response. The resulting failure can be a slide or a flow depending on the material characteristics and slope geometry. The resulting movements are due to internal causes and can occur after the trigger mechanism.
- Can occur in saturated, very loose granular deposits, very sensitive clays and loose loess deposits.

Cyclic Liquefaction

- Requires undrained cyclic loading where shear stress reversal or zero shear stress can develop (i.e. where in-situ static gravitational shear stress is low compared to cyclic shear stress).
- Requires sufficient undrained cyclic loading to allow effective confining stress to reach essentially zero.
- At point of zero effective confining stress no shear stress can exist. When shear stress is applied, pore pressure drops and a very soft initial stress strain response can develop resulting in large deformations. Soil will strain harden with increasing shear strain.
- Deformations during cyclic loading when effective stress is approximately zero can be large, but deformations stabilize when cyclic loading stops, unless pore pressure redistribution effects are large. The resulting movements are due to external causes and occur during the cyclic loading.
- Can occur in almost all sands provided the size and duration of cyclic loading are sufficiently large. For very dense sands the size and duration of cyclic loading will be large and hence, the condition of zero effective confining stress may not always be achieved.
- Clays can experience cyclic liquefaction but deformations at zero effective stress are generally small due to the cohesive strength at zero effective stress and deformations are often controlled by rate effects (creep).

Cyclic Mobility

- Requires undrained loading where shear stress is always greater than zero, i.e. no shear stress reversal develops.
- Zero effective stress does not develop.
- Deformation during cyclic loading will stabilize. The resulting movements are due to external causes and only occur during the cyclic loading.
- Can occur in almost any sand provided the size and duration of cyclic loading are sufficiently large and no stress reversal occurs. Can also occur in very dense sand with shear stress reversal, provided cyclic loading is not sufficient to cause zero effective stress to develop.

- Clays can experience cyclic mobility but deformations are often controlled by rate effects (creep).

PROPOSED FLOW CHART TO EVALUATE LIQUEFACTION

Figure 1 presents a suggested flow chart for the evaluation of liquefaction according to the above definitions. The first step is to evaluate the material characteristics in terms of strain softening or strain hardening response. If the soil is strain softening, flow liquefaction is possible if the soil can be triggered to collapse and if the gravitational shear stresses are larger than the ultimate residual or steady state strength. The trigger to cause collapse can be either monotonic or cyclic. Whether a slope or soil structure will fail and slide will depend on the amount of strain softening soil or limited strain softening soil relative to the strain hardening soil within the structure and on the brittleness of the strain softening soil. Dawson et al. (1993) have shown that at high effective stresses some strain softening granular soils appear to become less brittle with increasing confining stress. The resulting deformations of a soil structure with both strain softening and strain hardening soils will depend on many factors, such as: distribution of soils, geometry of structure, amount and type of trigger mechanism, brittleness of strain softening soil and drainage conditions.

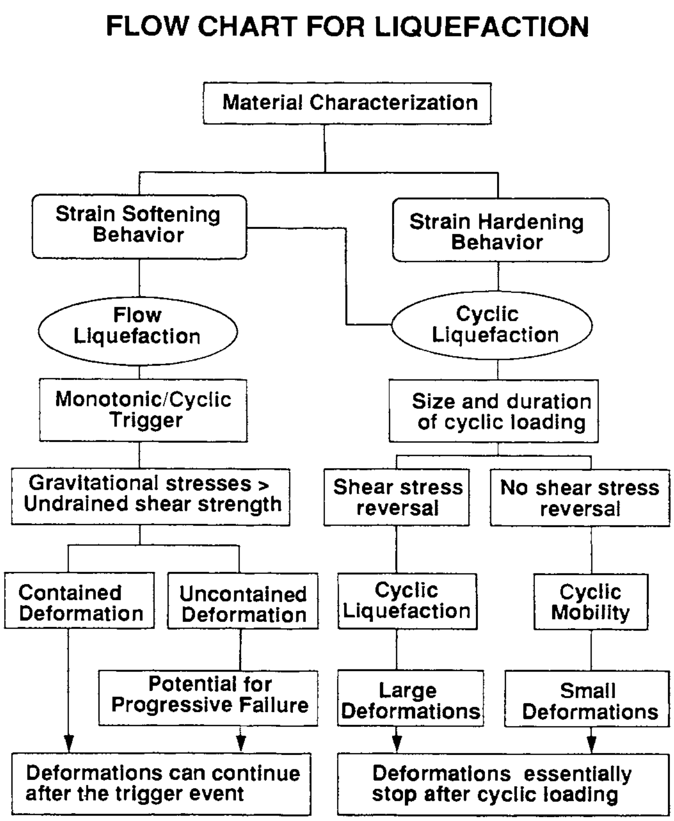

Figure 1. Proposed Flow chart for evaluation of liquefaction (After Robertson, 1994).

If the soil is strain hardening, flow liquefaction will not occur. However, cyclic liquefaction can occur due to cyclic (seismic) undrained loading. The amount and extent of deformations during cyclic loading will depend on the size and duration of the cyclic loading and on whether shear stress reversal occurs. If shear stress reversal occurs it is possible for the effective stress to reach zero and hence, cyclic liquefaction can take place. At the condition of zero effective stress large deformations can occur. If shear stress reversal does not take place or if the sand is very dense and it is not possible to reach the condition of zero effective stress, deformations will be smaller, hence, cyclic mobility will occur. Pore pressure redistribution can occur after cyclic liquefaction that can result in large deformations after the cyclic loading. Pore pressure redistribution generally takes place when a layer of low permeability soil is overlying the liquefied soil.

TEST SITES

Three sites have been selected for the initial phases of the CANLEX Project. The Phase I site is located on the Syncrude Canada Ltd. site near Ft. McMurray, Alberta (see Figure 2), and consists of hydraulically placed sand as part of the Mildred Lake Settling Basin. The Phase II sites are located in the Fraser River Delta near Vancouver, B.C. (see Figure 2) and consist of natural deposits of alluvial sand.

The Syncrude Canada Ltd. site was fully characterized during Phase I, and will be described in more detail in this paper. The Fraser River sites were fully characterized during Phase II, and will only be partly described in this paper.

Phase I Test Site (Syncrude Canada Ltd.):

The Syncrude Canada Ltd. surface mine, located near Ft. McMurray in northeastern Alberta, produced approximately 128 million tonnes of oil sand feed in 1993 generating approximately 280,000 tonnes of tailings solids daily. The Mildred Lake Settling Basin, commissioned in 1978, was designed as an all encompassing storage facility, accommodating the tailings slurry comprised of process water, sand, fine clay particles, and trace amounts of unrecovered hydrocarbons. The tailings slurry is pumped hydraulically via 600 mm diameter pipelines. Since start-up, the majority of tailings have been stored in the Mildred Lake Settling Basin, as shown in Figure 3.

Tailings sand is used to hydraulically construct containment dykes and supporting beaches of the storage facility, while the process water, released through the sand structures, is retained and re-introduced into the extraction process. This hydraulically constructed basin is currently comprised of about 600 x 10^6 m^3 of solids and contains approximately 280 x 10^6 m^3 water and fine tails in suspension. The fine tails, primarily clay particles,

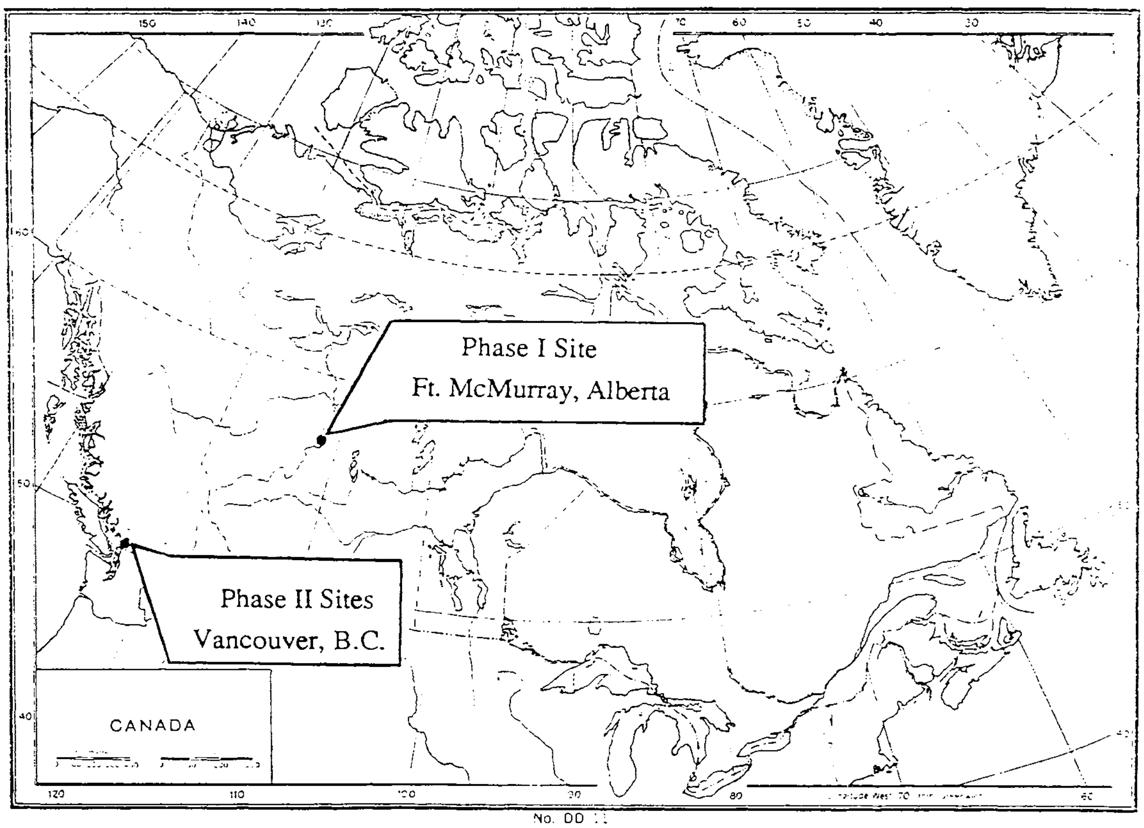

Figure 2 Site location for CANLEX Phase I and II sites.

accumulating in the basin are allowed to settle and consolidate over time. The settling basin profile, as illustrated in Figure 4, includes a freewater layer, a consolidating fine tails zone, and a mature fine tails zone within the sand containment structure.

Two primary sand deposition techniques have been utilized to construct the settling basin containment structure: cell construction and overboarding. Cell construction techniques are utilized to develop the 17 km long perimeter dyke. Accompanied by mechanical compaction, a dense containment structure is developed. The majority of tailings is deposited through overboarding forming beaches within the perimeter dyke. The beaching techniques employed yield two modes of deposition; Beach Above Water (BAW) and Beach Below Water (BBW). Coarse tailings are the first to settle out from the point of discharge on the way to the water body forming a BAW deposit of medium density. The finer materials eventually reach and settle in the water body forming a BBW deposit of lower density. During the early stages of construction of the settling basin, some sand size tails were deposited below water to form BBW deposit of lower density in small localized parts of the basin system. These looser sand deposits, have now been covered by both BAW and cell construction and can be up to 40 m below existing ground surface. It was these looser sand deposits that were to be characterized by the CANLEX Project.

In June 1993, an initial field screening program was conducted through ConeTec Investigations Ltd. on the Mildred Lake Settling Basin to verify the exact test site location. A site was located within Cell 24 along the west portion of the perimeter dyke. The ground conditions comprise about 27 m of dense clean fine sand from BAW and cell construction. From 27 m to 40 m the sand is looser probably due to BBW construction. The phreatic surface during site characterization was at a depth of 21 m, hence the vertical effective stress from 27 m to 40 m varies from about 400 kPa to 600 kPa. The target zone for the CANLEX site characterization was from 27 m to 37 m.

A circular test site plan was developed, as shown in Figure 5. At the center of the test area in-situ freezing was carried out to obtain undisturbed samples. Around the center, at a radius of 5 m, various in-situ testing and sampling techniques were carried out. Testing included; seismic cone penetration tests (SCPT), standard penetration tests (SPT) with rod energy measurements, self-boring pressuremeter tests (SBPMT) and geophysical logging. High quality conventional sampling using a fixed piston sampler and Christiansen double tube core sampler was also carried out. The penetration testing (SCPT and SPT) was carried out through the University of British Columbia under the direction of Prof. R.G. Campanella. The field program was

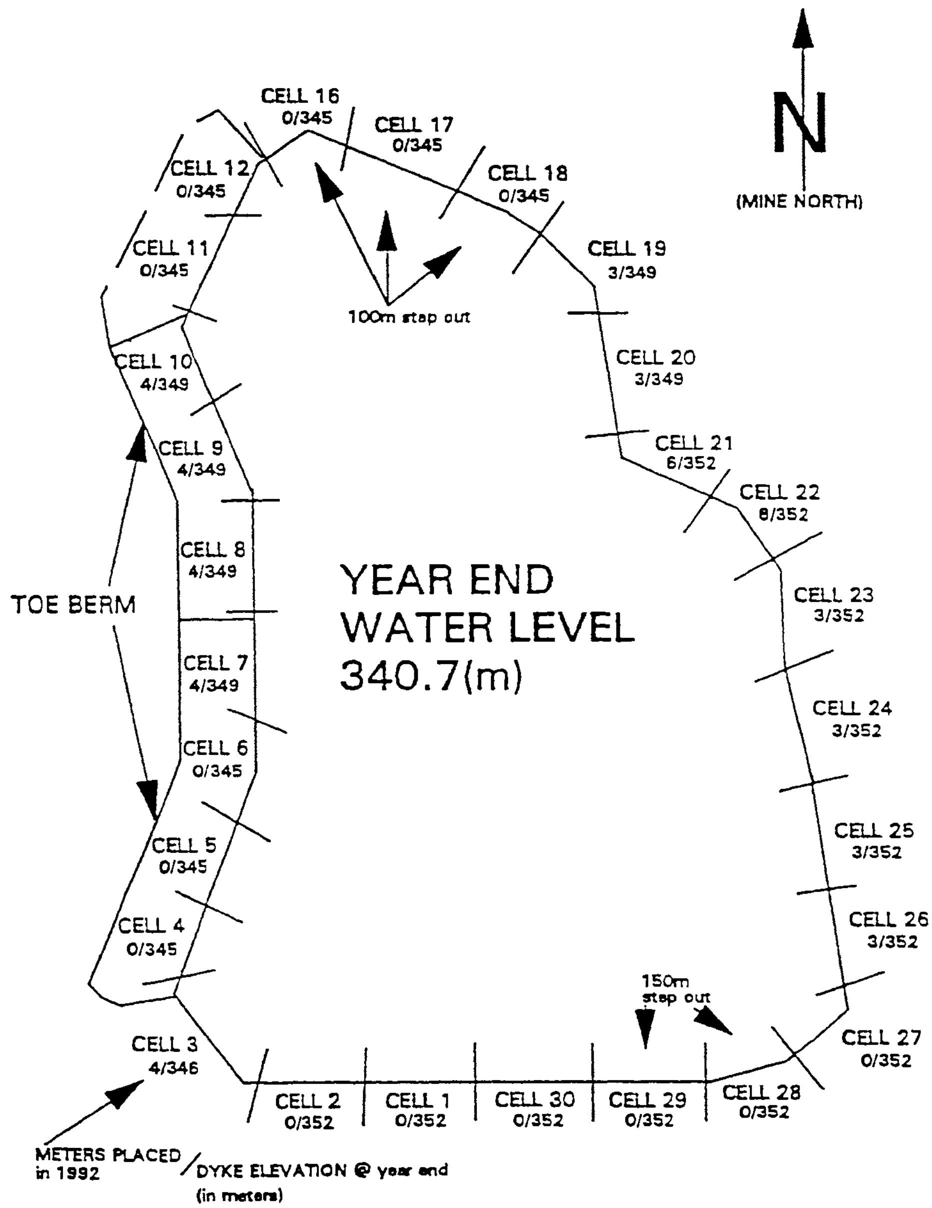

Figure 3. Site plan of Mildred Lake Settling Basin at Syncrude Canada Ltd. site.

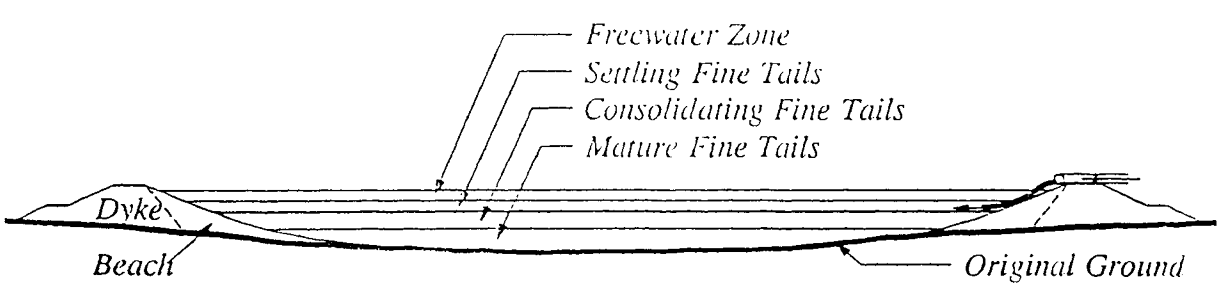

Figure 4. Schematic of Mildred Lake Settling Basin cross section.

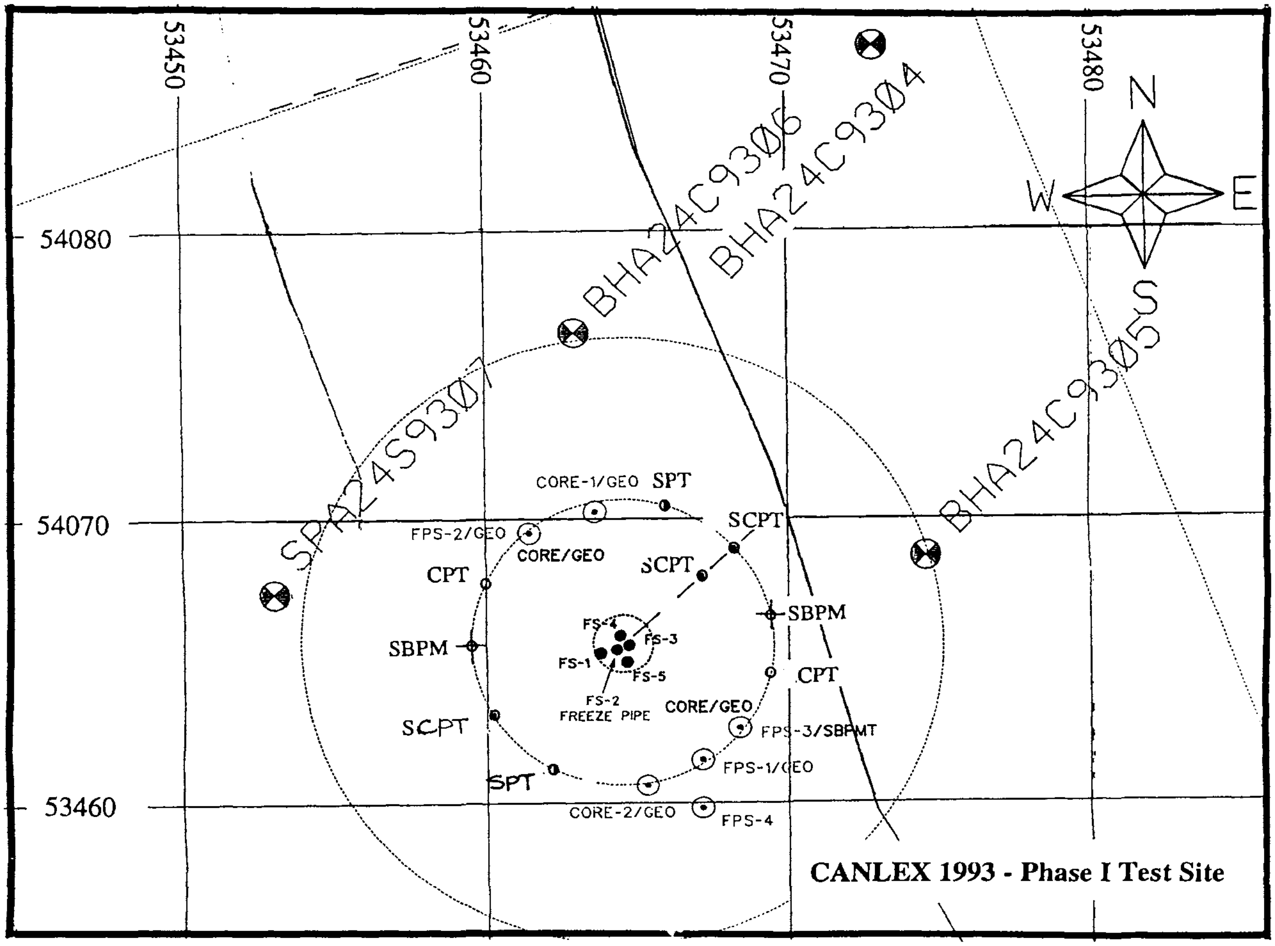

Figure 5. Phase I site plan showing test locations.

coordinated by Mr. M.P. Davies utilizing a Cone Testing Vehicle supplied by the B.C. Ministry of Transportation and Highways. The self-boring pressuremeter testing was carried out by Dr. J.M.O. Hughes. The SBPMT's were carried out in two parts; an initial evaluation of the pressuremeter technique was carried out in November 1993 followed by a more detailed testing program in March, 1994. Figure 6 shows a summary of the normalized penetration resistance values from the CPT's performed around the central in-situ freeze zone. The penetration resistance values have been normalized with respect to the effective vertical overburden stress and plotted against elevation. The elevation of the ground surface at the time of the field work was approximately 352 m.

The geophysical tests were completed in November 1993 and provided detailed profiles of interpreted density, moisture content and degree of saturation with depth (Plewes et al., 1988). Geophysical tools used in the field program included slim-line compensated gamma-gamma density, natural gamma, compensated neutron, resistivity and caliper measurements. The geophysical tests were carried out through Century Geophysical under the supervision of Dr. A. Küpper (AGRA Earth and Environmental Limited).

The conventional sampling was also carried out in the November field program with specially modified Fixed Piston Sampler and a Christiansen double tube core sampler under the supervision of H.D. Plewes (Klohn-Crippen Consultants Ltd.). A total of 17 piston samples of approximately 0.5 m length, totalling about 15 m of continuous core were successfully retrieved. Samples were frozen at the ground surface to improve handling and transportation. Details of the geophysical and conventional sampling procedures are given by Plewes et al., (1988) and Plewes et al., (1993).

Phase II Test Sites (Fraser River Delta)

A careful review and classification of over 14 potential sites was carried out by a team of engineers from the Vancouver area. Many different sites were reviewed and a short list developed. Two sites were selected for detailed site characterization. As with the Phase I site, a clear selection criteria was developed in order to select the most

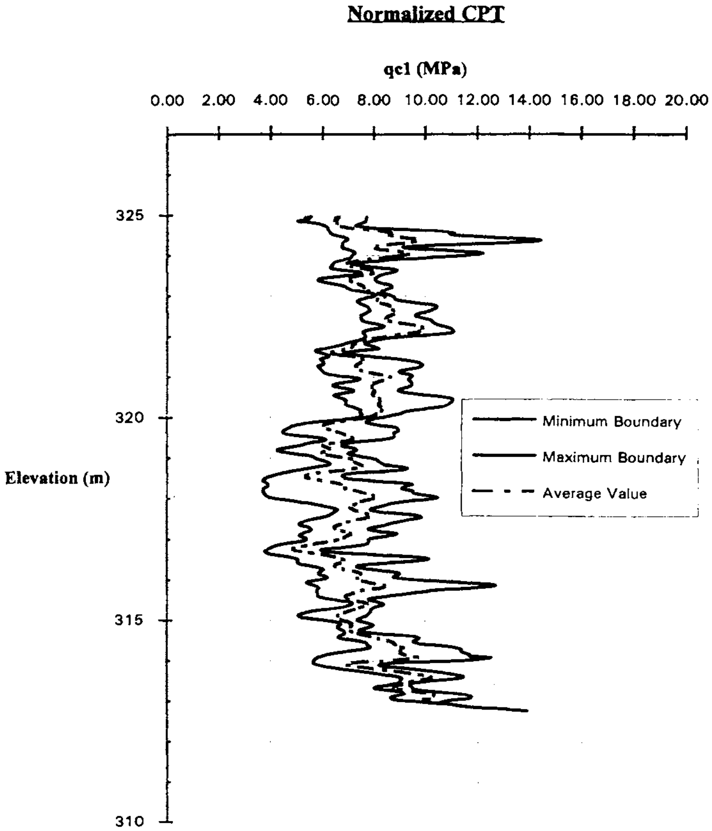

Figure 6. Normalized CPT penetration resistance profiles at Phase I test site.

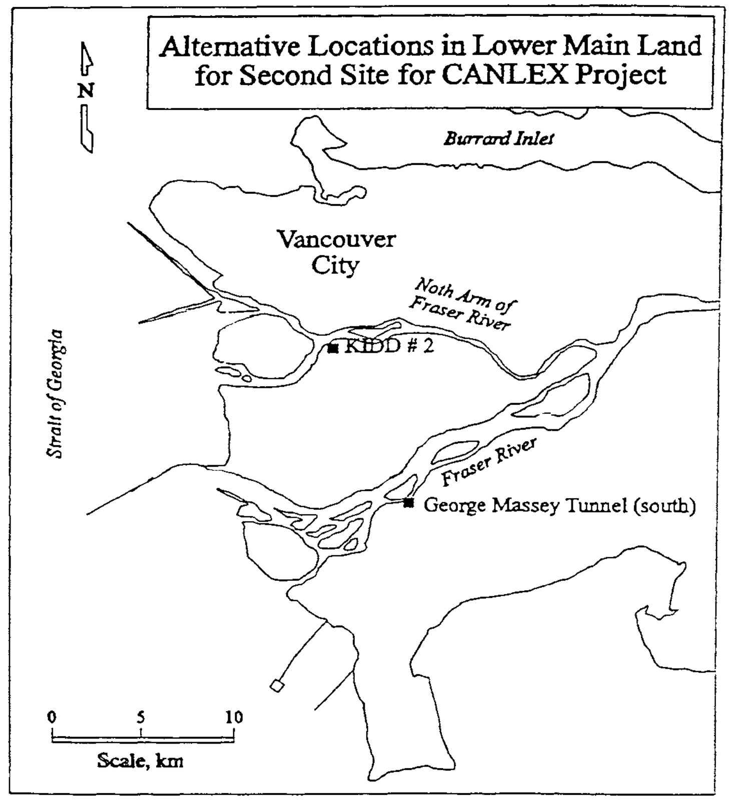

Figure 7. Location Plan for Phase II sites near Vancouver, B.C.

suitable sites. The selection was based on criteria such as, availability, security, size and the representativeness of the sand deposit.

The selected sites, located in the Fraser River delta, are the KIDD 2 substation in north Richmond, owned by B.C. Hydro and the Massey Tunnel (south portal) in Delta, owned by the B.C. Ministry of Transportation and Highways, as shown in Figure 7.

The subsoil's at the KIDD 2 substation are typical of those found in the Richmond area, comprising a surficial cover of organic silt and clayey silt to a depth of about 4 m over a loose to medium dense clean sand which extends to a depth of 22 m. The sand is generally considered susceptible to cyclic liquefaction under the design earthquake for the region using current methods.

The Massey Tunnel (south) site consists of a relatively loose, clean sand from 6 m to approximately 32 m below ground surface. This deposit is also considered to be susceptible to cyclic liquefaction. At both the KIDD 2 and Massey Tunnel sites the groundwater level is approximately 1.5 m below ground surface. Similar circular test areas have been established at both Fraser River delta sites.

The sand at both Phase II sites in the Fraser River delta are considered to be part of the same extensive distributary channel sand complex (Monahan et al., 1993).

IN-SITU FREEZING

One of the primary objectives of the CANLEX Project is to develop and enhance the process of in-situ freezing to obtain undisturbed samples. Careful laboratory studies have shown that sand can be frozen without causing disturbance to the density or fabric (e.g. Sasitharan et al., 1994). Davila et al. (1992) has shown that sands with fines can be frozen without disturbance provided the amount and type of fines are below certain limits. The disturbance due to freezing appears to be controlled primarily by clay minerals present in the fines. If the fines contain no clay minerals, the sand can have a large fines content and still be frozen without disturbance. However, small amounts of highly active clay minerals can cause disturbance during freezing. Davila et al. (1992) proposed guidelines to define the limits for undisturbed freezing based on the amount and type of fines.

During the summer of 1993 a team from the University of Alberta, under the supervision of Professor D.C. Sego, evaluated and enhanced the in-situ freezing technique to "target freeze" certain depths around a single freeze pipe.

During the November 1993 field program, this team carried out in-situ freezing to obtain undisturbed samples from the target depth of 27 m to 37 m at the Phase I test site. In-situ freezing was carried out from a central freeze pipe using liquid nitrogen. Radial freezing produced a column of frozen sand with a radius of 1 m. A total length of 20 m of frozen core was obtained using a 100 mm diameter CREEL core barrel. The in-situ freezing at the Phase I site represents a significant achievement due to the great depth of overburden (27 m), high ground temperature (approximately +12°C), and lateral ground water flow (approximately 0.25 m/day).

LABORATORY TESTING

The objectives of the laboratory testing program has been to determine the response of the sand to both static (monotonic) and dynamic (cyclic) loading. The role of the monotonic testing is to determine the state of the sand relative to its ultimate steady or critical state, and then to relate this back to the various in-situ test results.

In order to process all of the expected laboratory samples in a cost effective and efficient manner, and in keeping with the specific interests of the participants, testing is underway at laboratories at four different universities and one engineering consulting company. To provide some level of quality assurance within a given laboratory and among different laboratories, a testing protocol has been developed that describes the thawing, consolidation and shearing procedures. Each laboratory has been asked to test essentially identical frozen reconstituted samples of Syncrude sand to evaluate the proposed protocol. Laboratory testing is underway and results will be published in subsequent reports and papers.

LIQUEFACTION EVENT

To evaluate our ability to predict liquefaction response of sand structures a full scale liquefaction event is planned for Phase III of the CANLEX Project. Currently, millions of dollars are spent to retrofit structures in seismic areas due to expected liquefaction. Major concerns with the design process relate to the level of disturbance required to trigger liquefaction, the resulting residual undrained strength and the likely deformation. Two possible events have been studied:

Event 1: Flow Slide

This would involve loose sand and a steep slope such that when triggered flow liquefaction will result in a flow slide and provide a measure of residual strength. The trigger could be static or dynamic.

Event 2: Lateral Spread

This would involve either loose or medium dense sand and a more moderate slope such that when triggered cyclic liquefaction would result in lateral spreading rather than flow. The trigger must be dynamic, i.e., blasting, impact or vibroseis.

A feasibility study has been completed that has selected a potential test zone in an area of the future toe berm along the north side of the Syncrude Settling Basin. A flow slide event has been given highest priority with a static trigger. The trigger will likely be induced by rapid construction of a berm over about 10 m of very loose, saturated Syncrude sand. Approximate dimensions of berm and test cell will be confirmed by numerical and physical modeling.

To evaluate and plan the proposed event, preliminary centrifuge testing has been conducted in 1993 by C-CORE under the direction of Dr. R. Phillips and Prof. P.M. Byrne. An 18 degree submerged slope of Syncrude sand was rapidly loaded to trigger a liquefaction flow slide. This represents, what is believed to be, the first static liquefaction failure carried out in a centrifuge. Full details are given by Phillips & Byrne, (1994).

The results of the centrifuge tests are currently being analyzed to assist in the event planning. Analyses are underway by Prof.'s Finn and Byrne at the University of British Columbia, Prof.'s Chan, Robertson and Morgenstern at the University of Alberta and Dr. Gu at EBA Engineering Consultants Ltd.

Additional centrifuge tests are planned to clarify details of the proposed event.

SUMMARY

This paper has briefly presented a summary of the work completed during Phase I and part of Phase II of the CANLEX Project. Three test sites have been selected and site characterization has been essentially completed. Characterization has included in-situ testing, such as, CPT, SPT, SBPMT and geophysical logging. Samples have been obtained using high quality conventional techniques with a fixed piston sampler as well as in-situ freezing techniques. Feasibility planning which has included preliminary centrifuge testing has been completed for a full scale liquefaction event.

This work has been a collaborative effort between industry and universities in Canada.

ACKNOWLEDGMENTS

This work was partly supported by CANLEX (Canadian Liquefaction Experiment), which is a project funded through a Collaborative research and development Grant from the Natural Science and Engineering Research Council of Canada (NSERC), B.C Hydro, Quebec Hydro, Syncrude Canada Ltd. and Suncor Inc. The collaboration includes the geotechnical consultants, AGRA Earth and Environmental Limited, Klohn-Crippen Consultants Ltd., Golder Associates Ltd., Thurber Engineering Ltd. and EBA Engineering Consultants Ltd., as well as faculty and students from the Universities of Alberta, British Columbia, Laval, Carleton and Sherbrooke.

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