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EVALUATION OF STATIC LIQUEFACTION POTENTIAL USING UNDISTURBED SAND SPECIMENS

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SUMMARY

This paper presents the test protocols, and results of a study aimed at evaluating the liquefaction resistance of in-situ sands using laboratory tests. “Undisturbed” sand specimens were obtained using in-situ ground freezing techniques at four sites in Western Canada. The frozen specimens were thawed and in-situ stress conditions were reinstated prior to undrained shear in the laboratory. Different testing techniques were evaluated to determine the optimal test protocol that would ensure minimal disturbance to the test specimen during thawing and consolidation. It is shown that thawing at a small hydrostatic effective stress, and then consolidating to the in-situ effective stress state results in minimal disturbance compared to thawing at in-situ stresses.

Monotonic undrained response of the undisturbed sands was dependent on the loading mode. Almost all undisturbed specimens were dilative, and thus strain hardened under triaxial compression mode of loading. However, sands at looser density states strain softened under simple shear and triaxial extension modes of loading. Such direction dependent response is a manifestation of the inherent anisotropy present in natural alluvial sands.

INTRODUCTION

Undrained response of a sand under a given loading mode is dependent on its fabric, in addition to the initial stress, and density state [1]. Different natural deposition processes, (or construction techniques) yield different soil fabric. Duplicating the exact in-situ fabric in the laboratory is a difficult, and almost impossible task. In addition, the influence of aging and cementation are difficult to reproduce in the laboratory in a timely manner. As a result, it is generally considered that a confident assessment of the behaviour of natural soils may be made in the laboratory only if tests are carried out on undisturbed in-situ soils.

The soil deposits that are most prone to liquefaction are generally granular and cohesionless. Therefore, the conventional sampling techniques will not yield good quality samples in these deposits.

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In-situ ground freezing is the only viable technique that can provide relatively undisturbed samples in granular materials. The frozen specimens will have to be thawed, and in-situ effective stresses reinstated in the laboratory prior to testing. It is essential that the thawing and consolidation process does not significantly alter the structure of the material, and thus preserve the “undisturbed” nature of the soil specimen.

Liquefaction susceptibility of uniform, loose sand deposits at four different sites in Western Canada are evaluated using triaxial and simple shear tests in this study. This paper presents the undrained triaxial and simple shear test results, and describes the various testing protocols evaluated for thawing and consolidating the specimens to the in-situ anisotropic stress state. The strains induced during thaw and consolidation of the frozen specimens were monitored, and the degree of disturbance among different methods are compared.

IN SITU GROUND FREEZING & UNDISTURBED SAMPLING

The potential for obtaining high quality undisturbed samples using in-situ ground freezing has been recognized for a long time [2]. However, its usage did not gain widespread acceptance because of the various practical issues, and due to the relatively high costs. Site characterization and sampling of granular materials become essential when attempting to manage the liquefaction hazard. Primary interest in the phenomenon of liquefaction was triggered following the 1964 earthquakes in Niigata and Alaska. As a result, various studies have been conducted during the last two to three decades, both to refine the in-situ ground freezing techniques [3, 4], and to assess the effects of freeze-thaw cycles on the response of soils [5, 6].

The freezing process should ensure that soils remain “undisturbed” during sampling. The void ratio, and fabric of the sand would be altered if the soil particles rearrange as the pore water freezes into ice. The freezing process is associated with about 9% increase in volume, and in order to avoid disturbance to the soil skeleton, the excess volume must be removed from the pore space. Unlike in clayey soils, the attractive forces between soil particles and water molecules are extremely weak, and the permeability is relatively high in granular soils. As a result, much less energy is required to expel pore water out of the void space than to move apart the sand grains under the confining stresses. Therefore, under appropriately slow rates of freezing the excess volume of water would be driven out ahead of the freezing front in order to accommodate the volume increase associated with water freezing into ice, without disturbing the sand particle assembly.

All sampling techniques involve some form of stress release, and subsequent reinstatement of the in-situ stresses. Even if the in-situ stresses are restored with no change in soil skeleton or moisture content (thus preserving the fabric and the void ratio), the process imparts an additional stress history on the soil specimen. This additional stress history implies that no sampling technique can yield a perfectly undisturbed specimen which is equivalent to the soil element in-situ. However, the extra stress history imparted by the sampling process does not significantly influence the response. A sample may be considered “undisturbed” if its mechanical characteristics are expected to be identical to that of the in-situ soil. The term undisturbed specimen is used in this paper to refer to high quality samples retrieved using in-situ ground freezing.

Ground freezing and sampling at different sites were undertaken as part of the Canadian Liquefaction Experiment (CANLEX) research program. CANLEX project was a co-operative research endeavour with several University and Industry partners. Uniform loose sand deposits were identified based on in-situ tests and undisturbed sand samples, from depths ranging from 4m to 37m, were obtained using ground freezing techniques. Ground freezing procedures employed at each of the CANLEX sites together with the details of coring and trimming of specimens are explained in detail in Hofmann [7]. The behaviour of undisturbed sands specimens obtained from four different sites during the first three phases of the CANLEX project are presented in this paper. Phase I and III sites are located at an oil sand mine operated by Syncrude Canada Ltd., north of Fort McMurray in Alberta (Mildred Lake, and J-pit sites respectively). Two natural sand deposits in the Fraser River Delta, just south of the city of

Vancouver were investigated in Phase II. Phase I and Phase III sands are similar except for the deposition environment; both are rounded sands with about 12% fines, and are predominantly quartz. The Phase II Fraser Delta sands are sub rounded with a mineral composition of 40% quartz, quartzite and chert, 11% feldspar and about 45% unstable rock. The index properties, and other characteristics of the soil deposits are shown in Table 1.

Table 1: Properties of the undisturbed sands (modified after Fear [8])

		Phase I	Phase II		Phase III
		Mildred Lake	Massey	Kidd	J-Pit
Average Particle size D_{50} (mm)		0.2	0.3	0.35	0.2
Uniformity Coefficient, Cu		2.2	2.1	2.5	2.2
Fines Content (%)		~ 12	< 5	< 5	~ 12
e_{max} (ASTM D-4254)		0.958	1.102	1.061	0.901
e_{min} (ASTM D-4253)		0.522	0.610	0.703	0.579
Approximate age		30 years	200 years	4000 years	1 month
Depth (m)		27 to 37	8 to 13	12 to 17	3 to 7
Water Table (m)		21	1.5	1.5	0.5
in-situ test indices: mean (standard deviation)					
SPT $N_{1,60}$		18.2(3)	10.3(3.8)	17.2(4.8)	3.4(2)
CPT	q_{c1} (MPa)	7.5(1.7)	5.3(1.0)	6.5(1.8)	2.4(1.5)
	F (%)	0.73(0.15)	0.40(0.09)	0.37(0.05)	0.87(0.33)
Vs_1 (ms^{-1})		156(20)	168(6)	177(5)	127(3)

THAWING AND RESTORATION OF IN-SITU STRESS STATE

The frozen sand specimen has to be thawed, and in-situ stresses should be reinstated prior to testing in the laboratory. It is crucial that the process of thawing and/or restoring the in-situ stresses does not cause excessive disturbance to the specimen. Ideally all in-situ conditions including stress state, void ratio, fabric and stress history that prevailed prior to freezing would get restored in the laboratory in a truly undisturbed specimen. The most effective means of freezing sand with least disturbance is to invoke radial freezing outwards at a rate slow enough to expel the excess volume of water. The thermal conditions during thawing of the laboratory specimens would rarely facilitate such a unidirectional phase change. The specimen thaws radially inwards and axially from the pedestals during thawing.

In addition to the tests carried out at the University of British Columbia (UBC), tests on undisturbed samples were carried out at the laboratories in University of Alberta (UoA), Laval University and Klohn-Crippen Consultants Ltd. Table 2 summarizes the scope of the CANLEX laboratory testing program during Phase I, II & III. Several procedures have been proposed for thawing the frozen sand specimens

[6, 7, 9]. Obviously, the method that would result in minimal disturbance to the specimen would be the preferred technique for testing frozen specimens. In general, changes in void ratio would be the ideal indicator of specimen disturbance. However, measuring the actual void ratio change during the thawing process is not practical because of the difficulty in directly tracking the actual volume change during thaw. Therefore, the change in the height of the specimen, which can be measured with much greater confidence was considered as the most suitable index of the degree of disturbance.

Table 2: Laboratory tests on undisturbed sand specimens (CANLEX Phase I, II & III)

Phase	Test Type	Laboratory				Total
		UBC	UoA	Laval	Klohn Crippen	
Phase I	Triaxial	8	11	8	2	29
	Simple Shear	8	-	-	-	8
Phase II	Triaxial	21	-	12	-	33
	Simple Shear	26	-	-	-	26
Phase III	Triaxial	23	-	-	-	23
	Simple Shear	13	-	-	-	13
Total Tests (Phases I, II & III)		99	11	20	2	132

Frozen Specimens: Setup & Thaw

The setup and thawing process should be devised to minimize the disturbance to the soil. No uncontrolled thawing should occur during the setup phase, and adequate pore water should be supplied during thaw to fill the volume deficiency caused as ice melts into water. The loading caps, porous stones and the membranes used in testing frozen specimens were kept in the refrigerator together with adequate amounts of de-aired water. The drainage reservoir was filled with cold de-aired water so that the specimen would not begin to thaw when connected to the drainage reservoir. The undisturbed specimen was removed from the dry iced storage compartment in the freezer and setup in the apparatus using the cold loading caps, porous stones and membrane. A vacuum of about -80kPa was applied through the cold water drainage reservoir, immediately after sealing the membranes to the loading caps to remove the air trapped between the specimen and the membrane and porous stones. The cell was assembled and filled with cold water, and a cell pressure of about 20kPa was applied with the drainage reservoir open to the atmosphere. The vacuum on the specimen was then released. The entire setup procedure was streamlined, and the entire process (from the removal of the specimen from the freezer to the application of cell pressure) was completed within about six minutes. Since all surfaces that the specimen contacts are essentially at close to 0°C, negligible thawing is expected to occur during the setup process. This was confirmed by continuously monitoring the height of the specimen, which remained essentially constant during the setup process. The specimen was then allowed to thaw under a controlled room temperature of 23°C for at least 18 hours. During this time, the specimen had free access to water at both ends in order to allow for compensation of the volume deficiency caused by the pore ice melting into water. The height change of the specimen from the beginning of setup to the end of thaw was recorded.

Following thawing under a hydrostatic stress of about 20 kPa, the specimen was consolidated to the estimated anisotropic in-situ stress state. All triaxial tests were conducted at a back pressure equal to the in-situ pore pressure. In-situ vertical stress was calculated from the unit weight of the sand and the water table depth. In-situ horizontal stresses were calculated using the coefficient of earth pressure at rest, K_o obtained from pressuremeter tests [10]. The vertical effective stress was initially increased from 20kPa (at constant horizontal effective stress) to reach a K_o = 0.5, and the specimen was subsequently consolidated along the K_o line. Consolidation along the K_o line should result in zero radial strain, if the specimen is perfectly undisturbed. Any discrepancies would indicate disturbance and/or an error in the estimated K_o.

DISTURBANCE DURING THAW & CONSOLIDATION

Ideally, the restoration of the in-situ state, by the thawing and the subsequent consolidation of the undisturbed frozen specimen should not cause any void ratio change. As noted earlier, this is impossible to achieve because of the unavoidable minor disturbances an in-situ frozen specimen is bound to experience during sampling and subsequent storage prior to testing. The in-situ stress state is normally anisotropic in natural fluvial and hydraulic fill deposits, with vertical effective stress generally higher than the horizontal. Ideally, the stress state should remain unaltered in the soil element after freezing, except that the pore water would have been transformed into pore ice. Tensile stresses would develop in ice when the specimen is removed from the ground, similar to the development of negative pore water pressure when sampling a saturated clay specimen. In contrast to pore water, ice can carry some shear stresses, at least temporarily in the early stages of sampling [11]. However, ice has an extreme propensity to creep under shear and therefore cannot sustain large shear stresses for long periods of time [12]. Thus, the stress state in the retrieved frozen specimen will approach a hydrostatic state as ice creeps. Since undisturbed specimens were in storage for several weeks prior to testing, the stress state prior to thawing would have been virtually isotropic due to the inevitable unloading of the in-situ shear stresses.

An accurate determination of the in-situ lateral stress is quite difficult. The coefficient of earth pressure at rest K_o is estimated to be about 0.5 at all four sites based on pressuremeter tests [10], and consequently undisturbed specimens were consolidated to a K_o of 0.5. Any deviations from the actual in-situ value of the lateral stress would inevitably result in some disturbance causing alterations of the void ratio. Singh et al. [5] have shown that freeze - thaw cycles do cause insignificant volume changes, but their evidence was confined to initially hydrostatically consolidated specimens only. It is essential to recognize that their evidence cannot be translated to soils frozen under an initially anisotropic stress state.

As pointed out earlier, the void ratio change during thawing and consolidation to the in-situ stresses would be the best index of the degree of disturbance in the undisturbed specimen. However, the measured volume change during thawing consists of several components and that corresponding to the volumetric strain of the specimen cannot be easily separated out. The measured volume change includes (1) the volume intake due to ice thawing into water, (2) volume change of the specimen and (3) the high volume compliance of the system because the specimen is set up in air. Thus a confident measure of the void ratio change during thaw is difficult to achieve unless some unverifiable assumptions are made to calculate the volume change of the specimen. The height change during thaw, on the other hand, can be confidently measured using a displacement gauge positioned along the loading ram connected to the loading cap of the specimen. The height changes are therefore regarded as a more reliable substitute index of void ratio changes.

Height change during thaw and consolidation

Undisturbed specimens in Phase I were sampled from Mine Tailings at depths ranging from 28 to 37m, and the calculated degree of saturation in the samples was between 85 ~ 95%. The material was similar in Phase III, but obtained from shallower depth, and the saturation levels were between 95~100%. Most of the Phase II specimens were found to be fully saturated. B- value measurements at the end of thaw under 20 kPa effective stress, but at the in-situ level of the pore pressure were very low, and confirmed that Phase I Mildred Lake sand was unsaturated. The measured B-values in Phase I sand at in-situ pore pressure level were typically in the range of 0.2 to 0.5. The unsaturated Phase I specimens were saturated by applying differential vacuum in the laboratory prior to shearing. Both the pore space and the cell pressure chamber were subjected to negative (relative to atmospheric) pressure. The effective stress on the specimen was maintained within the range of 20 to 25 kPa during the application of differential vacuum. The B-value measurements of Kidd and Massey sands established that both Phase II sands were fully saturated. B-value of better than 0.99 were recorded in these sands. The Phase III Syncrude sand gave B-values in the range of 0.92 to 1 with an average of about 0.98. Thus both Phase II and Phase III sands were tested without subjecting them to any saturation process, in contrast to Phase I sands.

Sands with Fines (Phase I & III)

The measured net height changes, during thawing and consolidation of specimens, indicate relatively larger disturbance in Phase I Mildred Lake sand. The average axial strain during the restoration of in-situ stresses was about 1.5% in this sand. Even though both Syncrude sands (Phase I Mildred Lake & Phase III J-pit) are essentially identical, restoration of in-situ stress state in Phase III Syncrude sand resulted in smaller height changes. The main difference between the in-situ state of the Phase I & III sands is the degree of saturation, and the age. It is believed that the large height and thus void ratio changes in Phase I specimens occur in part due to freezing in the unsaturated state and in part due to the saturation process.

The total height change during thaw and consolidation of the undisturbed specimen as a function of the degree of saturation is shown in Figure 1. Data are shown for both undisturbed specimens, and specimens frozen and thawed under controlled conditions in the laboratory. At a given degree of saturation, the height change in the undisturbed specimens may be noted to be higher than those that were subjected to a freeze-thaw cycle in the laboratory. This suggests that the in-situ sampling process has imparted some disturbance to the sands. It can also be noted that Phase I sand, which is essentially identical to the Phase III sand with regard to the mineral composition and gradation, underwent much larger height change, and therefore disturbance, apparently on account of its low in-situ degree of saturation. Fully saturated specimens underwent minimal height changes. This data supports the contention that partial saturation might be a cause of the disturbance, and suggests that freezing is more suitable for sampling fully saturated sands. When partially saturated soils are frozen, gas/air pockets are likely to exist within the frozen specimen, which might cause collapse during thawing.

Fully saturated, Clean sands (Phase II)

The height changes during thawing and consolidation of Phase II sands specimens amounted to an average axial strain of about 0.14%, which is about a tenth of the axial strain observed in Phase I sands. Unlike in Syncrude sands, where both thaw and consolidation generally resulted in compression, most Phase II sands experienced swelling during thaw at 20 kPa confining stress, and compression during subsequent consolidation to the higher in-situ stresses. Since the effective stress during thaw was smaller than the in-situ stresses, the specimens would rebound if they are high quality undisturbed specimens. Figure 2 shows the height change during thaw and subsequent consolidation of all fully saturated Massey sand specimens tested under triaxial loading. Specimen height increase was in the range of 0.1 to 0.4 mm during thaw in these tests. This could be viewed as an indication that the undisturbed specimens are of high quality. The subsequent application of the in-situ stresses will cause compression, and in the ideal case, the magnitude of the initial rebound would be approximately equal to the compression during consolidation, because hysteresis during a small unload/reload cycle is small. The total height change during thaw and consolidation Massey sands ranges from about +0.1 to -0.2 mm. This small overall height change is an indication that relatively undisturbed samples of saturated sands can be obtained using in-situ ground freezing, and by allowing them to thaw under a small effective stress prior to reinstating in-situ stresses.

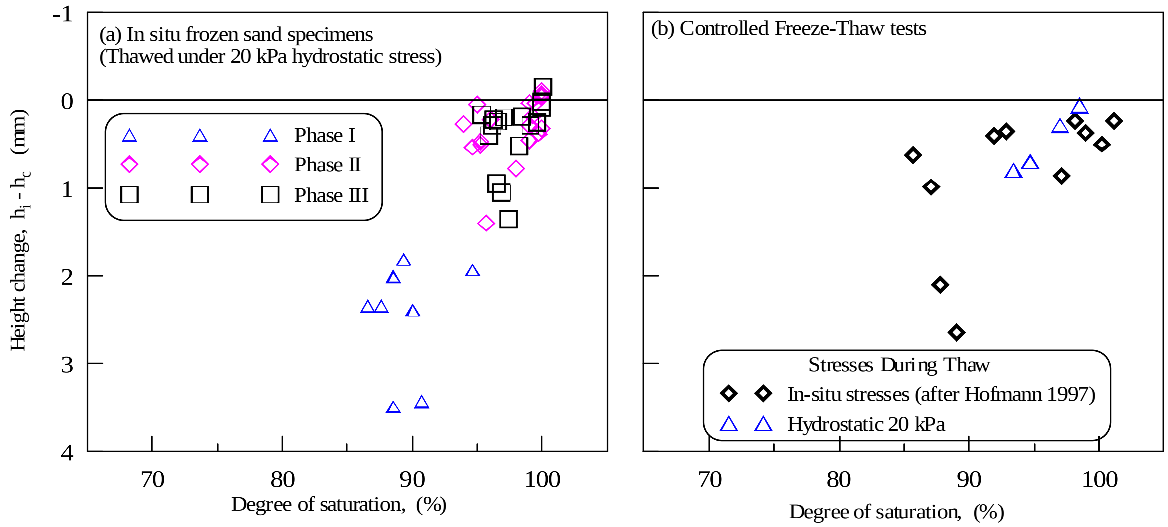

Figure 1: Height change during thaw and consolidation in Mildred Lake sand

Alternate Thawing Protocol

Tests conducted at University of Alberta used an alternate technique to thaw the frozen specimens, and the details are described in Hoffman [7]. The initial setup process was quite similar to that described earlier, but the stress conditions during thaw were drastically different. Porous stones, membranes and loading caps were all kept in a cooler, but the specimens were setup in a cold room, and water-glycol solution was used to fill the cell. Following the initial setup, specimens were moved to the testing room at a temperature of about 20°C, and 90% of the vertical and horizontal in-situ stresses and pore pressure were applied to the specimen. A warm glycol solution was circulated through the base plate to thaw the specimen under drained conditions from the bottom drainage line. Both the volume change in the reservoir connected to the pore space, and the specimen height changes were recorded. Tests at University of Alberta were conducted only during Phase I of the testing program. Specimens in Phase I were obtained from relatively deep locations (depths ranging from about 28 ~ 37m) and therefore the average initial stresses applied were about 600 kPa vertical and 300 kPa horizontal.

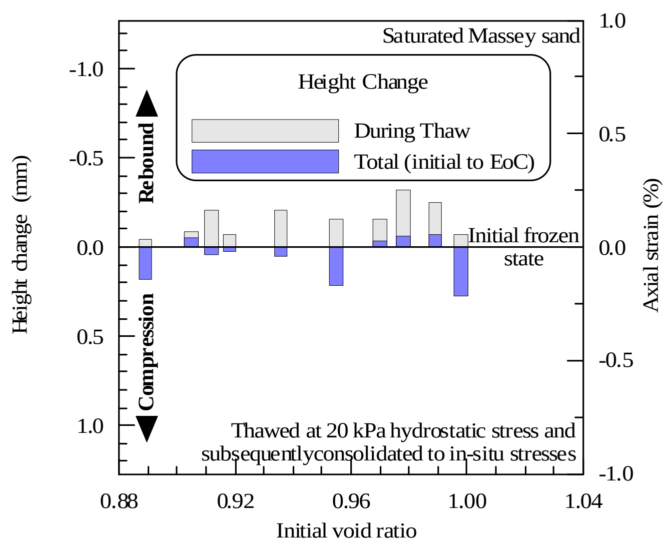

Figure 2: Height change during thaw and consolidation in saturated Massey sands

Evaluation of the thawing techniques

Comparative specimens of Phase I Syncrude sand were thawed both under a small hydrostatic effective confining stress (Method I) and under estimated in-situ stresses (Method II). Specimens thawed by Method I were subsequently consolidated to the estimated in-situ stress state. A summary of the total height changes, from the initial frozen state to that after the restoration of the in-situ stresses is given for both methods in Table 3. Both undisturbed in-situ frozen

Mildred Lake sand specimens, and specimens reconstituted in the laboratory and then unidirectionally frozen were used in these studies. Data from tests carried out at University of British Columbia [13], and University of Alberta [7] are shown in the table.

As noted earlier, the actual void ratio change of a specimen during thawing and the restoration of in-situ effective stresses cannot be measured without resorting to some assumptions. The height change measurements, on the other hand, can be reliably measured, and used as an alternative index of the void ratio changes. In this light, the data in Table 3 indicates that the specimens thawed under a small hydrostatic effective stress and then consolidated to in-situ stresses undergo lesser disturbance than those thawed under high in-situ stresses. Specimens reconstituted in the laboratory were well saturated compared to the undisturbed in-situ frozen sand specimens. The in-situ frozen specimens undergo much larger height change in both methods, partly due to the degree of saturation and partly due to the disturbances caused by sampling in-situ.

Table 3: Height change during restoration of in-situ stress state in Mildred Lake sand

	Method I		Method II[2]	
	undisturbed	reconstituted[1]	undisturbed	reconstituted
No of Specimens	8	6	5	2
Axial Deformation (mm)				
Minimum	1.81	0.06	0.70	0.50
Maximum	3.49	0.29	8.65	0.86
Average	2.06	0.20	2.85	0.68

[1]*partial data, and* [2]*data from Hofmann (1997)*

Frozen specimens were allowed to thaw under room temperature in Method I, and in Method II warm glycol solution was circulated through the base platen in an attempt to simulate unidirectional thaw. The objective was to reverse the unidirectional freezing process in-situ during thaw. An increase in the rate of axial thawing would be achieved by circulating warm liquids through the end platens. However, thawing in the radial direction cannot be completely prevented because of the temperature differences between the cell fluid and the soil specimen. Thus, unidirectional thawing is only an idealized scenario, and in reality the specimens would thaw radially inwards as well as axially from the end platens. As a result, a frozen sand core that continually decreases in size would exist in the middle of the specimen surrounded by the thawed sand. Such a rigid inclusion would induce shear stresses in the specimen, even under hydrostatic boundary stresses. Consequently a non-uniform stress distribution will occur within the specimen. The severity of this non uniformity will be greatly enhanced, if thawing occurs under large non hydrostatic stresses. In contrast, the magnitude of these non uniform stresses would be small if the specimen is thawed under small hydrostatic stresses as in Method I. The influence of these small non-uniformities on the measured mechanical behaviour would diminish after the specimen is subsequently consolidated to the higher in-situ stresses. Since specimens are not consolidated to higher stresses in Method II, the non-uniform stress history will persist, and could influence the measured response to a larger extent.

Numerical simulations of the thawing process using simple models indicate very high non uniform stress distributions if the specimens are thawed under anisotropic in-situ stresses, rather than under a small hydrostatic stress [14]. Analytical simulations also show that shear strains developed during thawing by Method II to be an order of magnitude higher than those developed in Method I [7]. Even though simplifying assumptions were made in the analyses regarding the complex constitutive relations of frozen soils, these results support the experimental observations reported in Table 3.

The data presented in Table 3 together with assumed radial to axial strain ratio were used to compute the void ratio change in earlier studies [7]. Radial strain was assumed to be equal to twice the measured axial strain for Method I, but zero for Method II. It was believed that the presumed unidirectional thawing of specimens in Method II would not induce any radial strains under in-situ stress levels. As noted earlier, unidirectional thawing is only an idealized scenario, and even in the theorized case of unidirectional thawing, the circumferential stress boundary of the specimen, with flexible membrane cannot also be deemed to be a strain boundary at the same time.

Figure 3 shows the measured axial and radial strains during consolidation (along the K_o line) of all undisturbed specimens thawed by Method I. The radial strain during consolidation is consistently smaller than the axial strain during K_o consolidation. The ratio of radial strain to axial strain, $\varepsilon_r/\varepsilon_a$ is also plotted in the figure. This ratio is very small for most of the specimens, and the few higher values are a reflection of the very low axial strains (as low as 0.02%) recorded in those tests, and does not imply large volumetric strains nor a significant change in void ratio. Several specimens in Figure 3 exhibit negative $\varepsilon_r/\varepsilon_a$ ratio ranging from 0.0 to about -0.3. It should be noted that an $\varepsilon_r/\varepsilon_a$ ratio of -0.50 would correspond to zero volumetric strain. In contrast to consolidation along K_o path, the $\varepsilon_r/\varepsilon_a$ ratio in hydrostatically consolidated sands is very large (often greater than 2) on account of the inherent anisotropy. Consolidation along the K_o line in itself should result in zero radial strain. However, the in-situ K_o is not exactly known and all undisturbed specimens were consolidated at a K_o value of 0.5. Small radial strain would therefore occur during consolidation, partly on account of the difference between this laboratory K_o value and the actual unknown in-situ K_o, and partly due to disturbances during sampling.

It can be noted in Figure 3 that almost all Syncrude J-Pit sand specimens exhibit negative $\varepsilon_r/\varepsilon_a$ values. This might be an indication that the in-situ K_o of the Syncrude J-Pit sands might be higher than 0.50. Nevertheless, the measured radial strains are very small in all these tests. The data in the figure demonstrates that assuming the radial strain to be twice that of the measured axial strain during K_o consolidation is not rational. Such an assumption leads to progressively erroneous results with increasing axial strain. For a given axial strain, the radically different assumptions as to the $\varepsilon_r/\varepsilon_a$ ratio result in volumetric strain five times higher for specimens thawed by Method I than by Method II. Thus even when the sand specimens undergo equal axial displacement under both methods, these assumptions lead to five time larger calculated void ratio changes in Method I. The calculated void ratio change using this scenario is therefore a manifestation of the assumptions, and is not indicative of the actual void ratio

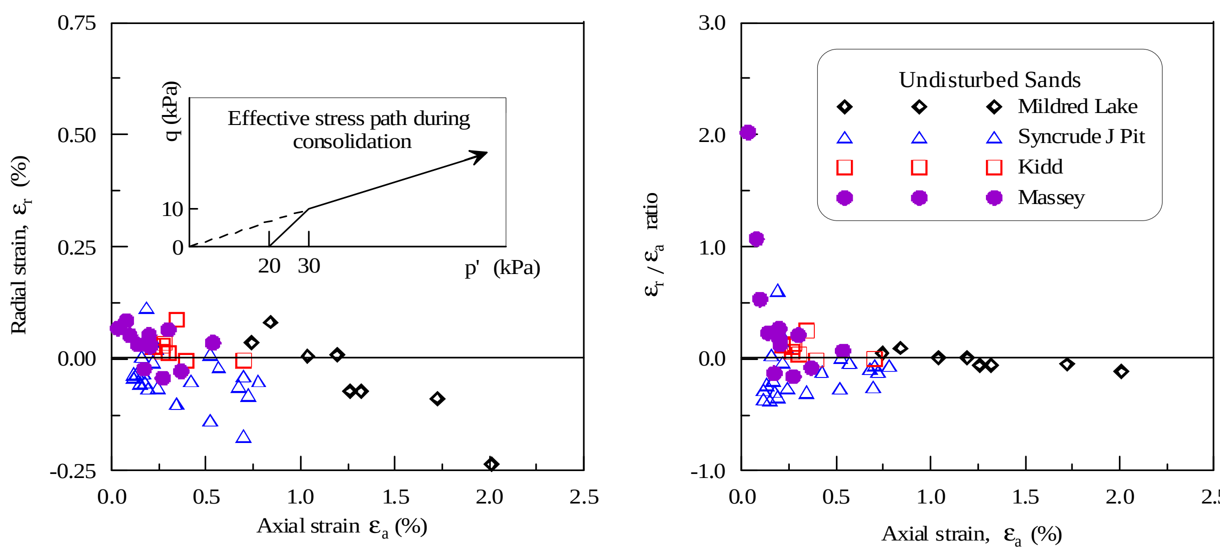

Figure 3: Axial and radial strain during reinstatement of in-situ stresses following thaw

change of the sand. The measured height change data, which is an indirect indicator of the void ratio change, suggest that Method I causes lesser disturbance to the specimen at the conclusion of restoring the in-situ effective stresses, compared to Method II.

EXPERIMENTATION

An NGI type simple shear apparatus [15] was used to carry out cyclic and post liquefaction simple shear tests on approximately 70mm diameter x 20mm high specimens. Simple shear specimens are enclosed in steel reinforced rubber membranes, and thus cannot develop any lateral strains. The pore space in simple shear is open to atmosphere, and constant volume conditions are enforced during the shearing phase. The change in total vertical stress during shear in a constant volume simple shear test equals the excess pore pressure generated in an equivalent undrained test [16]. Cyclic shearing was applied under stress controlled loading mode, and post liquefaction loading under displacement control.

Triaxial tests were carried out using a stress/strain controlled UBC cyclic triaxial device [17]. Triaxial tests were performed on approximately 63mm diameter by 125mm high specimens under displacement controlled loading. High resolution transducers and high speed 16 bit data acquisition system were used to accomplish confident measurements of strains with a resolution of about 10^{-4} and stresses with about 0.2 kPa, both in triaxial and simple shear tests. Schematic diagrams and other details of the simple shear and triaxial devices used in this study are given in Sivathayalan [13].

STATIC UNDRAINED BEHAVIOUR OF UNDISTURBED SANDS

Undrained Triaxial Response

Figure 4 shows the range undrained triaxial compression and extension response of undisturbed Kidd sand consolidated to the estimated in-situ effective stress states, and pore pressure. Only a selected number of tests representing the strongest and weakest extremes of the measured response are shown in the figure to improve clarity. None of the specimens exhibited true liquefaction type of response, neither in compression nor extension loading mode.

The Kidd sand is highly dilative in triaxial compression. Very little positive excess pore pressures developed in triaxial compression, and the strong strain hardening response resulted in very high strength associated with the generation of large negative excess pore pressure that eventually led to cavitation of the pore fluid. The essentially constant shear strength exhibited by the medium dense Kidd sand (e_c = 0.886) with strain, at about 900 kPa is a result of deformation at constant effective confining stress due to pore fluid cavitation. Cavitation of pore water has in effect rendered the undrained test a drained one. The sand would have exhibited an even higher strength, had the initial back pressure been higher than its in-situ value of 100 kPa. In such scenarios involving very dilative soils, the maximum

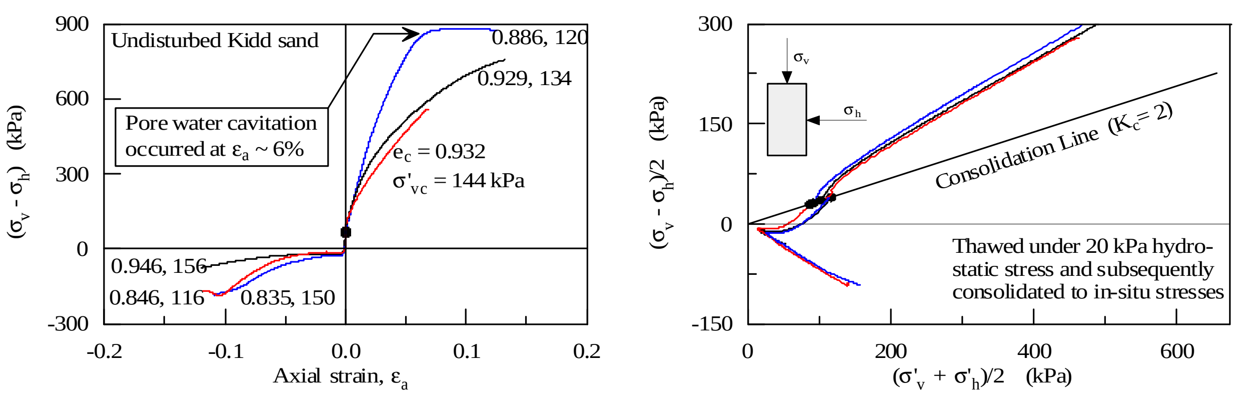

Figure 4: Undrained triaxial response of undisturbed Mildred Lake sand specimens

operating strength would be the drained strength. In contrast to triaxial compression loading, Kidd sand exhibits limited liquefaction [18] type of response in triaxial extension. The minimum undrained strength measured in triaxial extension is only a fraction of that measured in triaxial compression.

Similar triaxial data on the behaviour of Massey and Syncrude Mildred Lake and J-pit sands are shown in Figures 5, 6 and 7 respectively. All three sands exhibit direction dependent behaviour similar to that of Kidd sand noted above. All specimens under triaxial extension loading strain softened, and exhibited the limited liquefaction type of response. All but one specimen each in Massey & J-pit sand strain hardened in triaxial compression. The void ratio of the Massey sand specimen that exhibited limited strain softening deformation under compression loading is 0.926. This void ratio is much denser than those of several other specimens that strain hardened. This inconsistency in the data suggests that this individual specimen might have suffered larger disturbance during sampling. One Syncrude sand specimen exhibited marginally strain softening response in triaxial compression, but in triaxial extension this sand was extremely strain softening, with very low minimum undrained strength.

Specimens that were exhibiting dilative behaviour after reaching phase transformation would presumably realise steady state after all dilation is complete. However, the specimens would have to be subjected to very large strains to realise this state. The end restraint effects that are inherently present in all soil testing devices often restrict the range of strains that can be imposed in a laboratory. During these triaxial tests, the specimens remained essentially uniform up to an axial strain of 12-15% in compression and to about 10-12% in extension. Excessive bulging in compression and necking in extension was clearly noted beyond these strains. Several researchers have imposed axial strains in excess of 30% in conventional triaxial apparatus in order to find the ultimate or steady state strength of sands. Extreme caution is required in interpreting such data as the specimen at that large strain is far from being an element. The measured stresses and strains rather than being reflective of one state, are in reality now the average of excessively non uniform states of stresses and strains within the specimen.

The results presented above, comparing the triaxial compression and triaxial extension behaviour of four undisturbed sands clearly indicate that the behaviour of in-situ sands is direction dependent. Sands tested are rarely strain softening in triaxial compression, and never exhibit significant loss in strength beyond its peak in this mode even in their loosest state. Yet, they are almost always strain softening in triaxial extension. This direction dependent behaviour is a manifestation of the inherent undrained anisotropy in sands. The existence of such anisotropy in reconstituted sand specimens has been demonstrated by several researchers over the years [19-21]. The results presented above indicate that this direction dependence in undrained behaviour is predominant in natural alluvial/fluvial sands as well.

Constant Volume Simple Shear Response

Figure 8 shows the undrained simple shear behaviour of undisturbed Mildred Lake Syncrude sand specimens over a range of void ratio. The loosest specimen exhibits strain softening response, but with little loss of shear strength beyond the peak. The response gradually transforms from contractive into dilative as the relative density increases. The simple shear response is stronger than the triaxial extension response, but is weaker than triaxial compression. The orientation of bedding planes, which are horizontal on account of the gravitational deposition process in natural alluvial sands is responsible for this loading mode dependency. Comparison of the triaxial compression, extension, and simple shear behaviour clearly shows a systematic softening in the response as the direction of major principal stress moves from 0° to the vertical in triaxial compression to 45° in simple shear [22] and finally to 90° in triaxial extension. Vaid and Sivathayalan [23] have reported similar findings in specimens of this sand reconstituted by water pluviation.

The friction angle mobilised at quasi-steady state in strain softening specimens, and that at phase transformation in strain hardening specimens may be noted to be essentially identical in the Figure (SS/QSS/PT line in figure 8b). Similar equivalency of these friction angles has been noted in water pluviated sands [20, 23]. The undrained simple shear behaviour of the other undisturbed sands is also

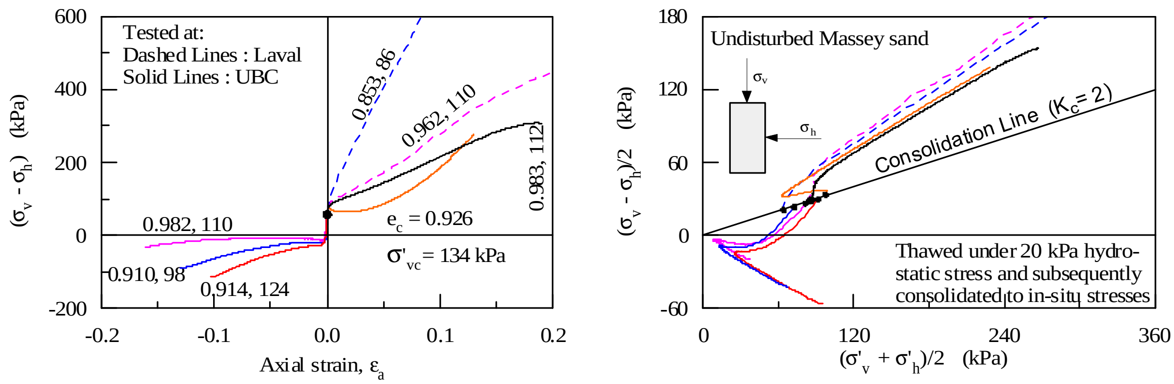

Figure 5: Undrained triaxial response of undisturbed Massey sand

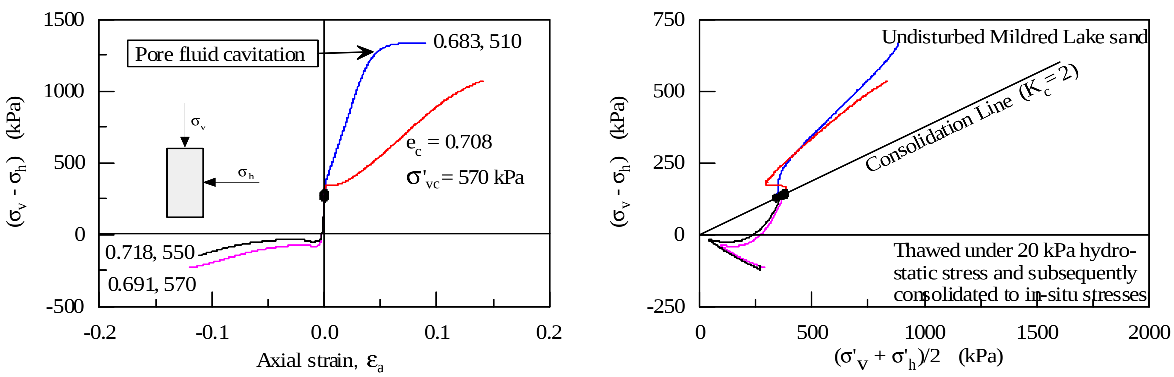

Figure 6: Undrained triaxial response of undisturbed Mildred Lake (Syncrude) sand

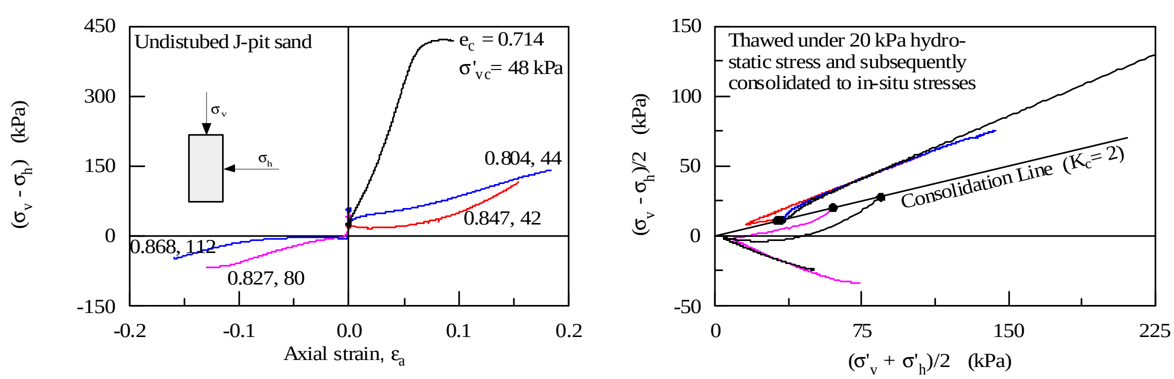

Figure 7: Undrained triaxial response of undisturbed J-pit (Syncrude) sand

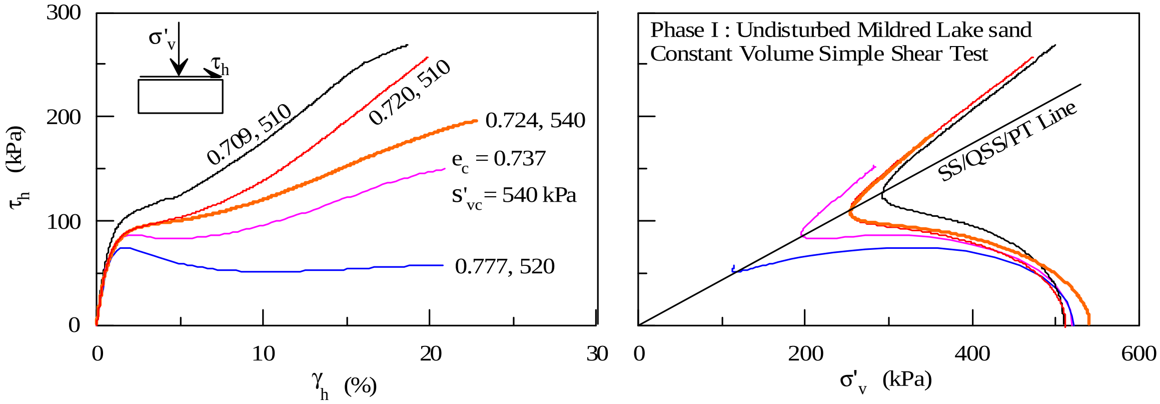

Figure 8: Range of undrained simple shear response in undisturbed Mildred Lake sand

essentially similar, in that they marginally strain soften in the loosest state, and increasingly strain harden as the void ratio decreases. These results clearly demonstrate the effect of stress path on the undrained response of in-situ sands

The friction angle mobilised at quasi-steady state in strain softening specimens, and that at phase transformation in strain hardening specimens may be noted to be essentially identical in the Figure (SS/QSS/PT line in figure 8b). Similar equivalency of these friction angles has been noted in water pluviated sands [20, 23]. The undrained simple shear behaviour of the other undisturbed sands is also essentially similar, in that they marginally strain soften in the loosest state, and increasingly strain harden as the void ratio decreases. These results clearly demonstrate the effect of stress path on the undrained response of in-situ sands.

CONCLUSIONS

An experimental study to evaluate the applicability of in-situ ground freezing technique to assess the liquefaction susceptibility of in-situ sands was undertaken as part of the CANLEX research program. Proper thawing techniques are essential to ensure the "undisturbed" nature of the frozen specimen. It is shown that thawing the specimen at a low hydrostatic confining stress, and then consolidating it to the in-situ stress levels would result in minimal disturbance. The sampling and testing process imparted very minimal disturbance in fully saturated clean sands, and somewhat larger disturbance in a saturated sand with about 12% fines. However, the techniques used in this study caused considerable disturbance if the in-situ sand is not fully saturated.

None of the sands exhibited the true liquefaction type of response. The monotonic undrained response of the sands from all four sites was dependent on the loading mode. Almost all specimens were dilative (and strain hardened) under triaxial compression mode of loading. However, sands at looser density states strain softened under simple shear and triaxial extension modes of loading. In general, compression response was the strongest, extension response the weakest, and simple shear response was in between. The direction of the major principal stress in a triaxial compression test is aligned with the direction of deposition under gravity, and the strongest response in this loading mode is a reflection of the material anisotropy. Similar direction dependent response has been noted in specimens reconstituted in the laboratory using pluviation techniques, and the present study demonstrates that such characterization is applicable to the alluvial sands in the field.

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