Strain Path Controlled Tests using a New Hollow Cylinder Torsional Shear Apparatus

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ABSTRACT
An automated Hollow Cylinder Torsional (HCT) shear device has been commissioned at Carleton University. This device is capable of both monotonic and cyclic testing under load and displacement controlled loading modes. It can follow prescribed stress/strain paths under generalised loading/drainage conditions. A comprehensive experimental study was carried out (using this device) to investigate the behaviour of sands subjected to different strain paths. The test results show that undrained state may not be the most damaging scenario under generalized loading conditions. Smaller minimum shear strengths (compared to the undrained strength) were measured when the boundary conditions resulted in expansive volume changes. The minimum shear strength of the soil during expansive volumetric strain deformation is further reduced, when the intermediate principal stress parameter increases from 0 to 1.

RÉSUMÉ
l'Université Carleton s'est procurée un appareil cylindre creux de torsion (HCT) de cisaillement. Ce dispositif peut effectuer des test monotoniques et cycliques en modes de charge ou de déplacement de charge. Il peux suivre les chemins de stress/contraintes prescrits, sous des conditions généralisées de charge/drainage. Une étude expérimentale utilisant ce dispositif a été réalisée pour étudier le comportement du sable dans différents chemins de contraintes. Les résultats des essais montrent que l'état non-drainé ne peut pas être le cas le plus dommageable dans des conditions générales de chargement. De plus petites forces de cisaillement minimum (par rapport à la force non-drainé) ont été mesurés lorsque les conditions limites ont menées à une importante expension de volume. Lors de déformation volumétrique du sol par contrainte, la force de cisaillement minimum du sol est réduite lorsque le parameter de stress intermédiaire principal augmente de 0 à 1.

1 INTRODUCTION

Laboratory assessment of soil behaviour is commonly carried out using conventional soil testing devices (e.g. triaxial, simple shear). These conventional devices do not normally represent the in-situ soil loading paths because they generally ignore the effects of intermediate principal stress and/or principal stress rotation/direction. In addition, the nature of the principal stress rotation in these devices may be completely different compared to that in-situ. A soil-testing device that has the capability to independently control the magnitude and directions of principal stresses enables confident assessment of the soil response in the laboratory.

The characteristics of soil deformability can be determined by stress parameters, such as the three principal stresses (major, intermediate and minor) and their inclinations to a set of orthogonal coordinate axes. In geotechnical practice, these stress parameters are commonly substituted by shear stress τ or deviatoric stress $\sigma_d = \sigma_1 - \sigma_3$, confining pressure σ'_3 or effective mean normal stress $\sigma'_m = (\sigma'_1+\sigma'_2+\sigma'_3)/3$, Intermediate principal stress σ_2 or intermediate principal stress parameter $b=(\sigma_2-\sigma_3)/(\sigma_1-\sigma_3)$ and Inclination of the principal stresses to soil deposition direction α_σ.

The HCT device generally permits independent control of four parameters. However, this device is not commonly used in the soil testing because of the complexities and the cost. A few researchers (e.g. Broms & Casbarian, 1965; Hight et al., 1983; Sayao & Vaid, 1988) have used this device in soil testing over the years to better understand the fundamental characteristics of soil deformation under drained and undrained loading. The new HCT device at Carleton University uses advanced control techniques to permit testing along different stress/strain paths, including the ability to conduct partially drained tests. This research program investigated the behaviour of sands under imposed expansive volumetric strain. This is intended to simulate post earthquake deformation conditions in situ

In the past, some earth structures (e.g. San Fernando Dam, Mochikoshi Dam) have failed during post loading deformation. The deformation failure of these soil structures has apparently occurred due to liquefaction, but not during the earthquake, but after the cessation of earthquake. The noted 'delayed liquefaction' has occurred due to void redistribution within the soil deposit. Void redistribution occurs in soil deposits on account of pore water migration due to the existence of piezometric pressure gradients.

Liquefaction failure may be critical in soil elements in sloped structures due to the presence of initial static shear. The presence of shear stress dictates the magnitude of the principal stresses and their direction in the soil element. Previous studies by Vaid and Chern (1985) have noted the influence of initial static shear stress on liquefaction susceptibility.

A few laboratory studies (e.g. Vaid and Eliadorani, 1998, 2000; Gananathan, 2000; Sivathayalan and Logeswaran, 2007, 2008) were carried out to further understand such post loading deformation behaviour of soils. Those studies were very limited in scope (all were confined to axisymmetric loading). But, most of the post-loading deformation problems in-situ do not correspond to axisymmetric loading. Deformation condition in dams, for example, is closer to plane strain with possible stress rotation. Variations in the magnitude of the intermediate principal stress and directions of the major/minor principal stresses may significantly affect the response, and the simpler axisymmetric tests reported in the literature may not provide proper insights into the mechanisms of deformation in such problems. This research will assess the response of sands under generalized loading conditions that involve simultaneous changes in pore pressure and pore volume to better understand the nature of void redistribution under generalized loading.

2 EXPERIMENATION

Detailed description of the new HCT device, its data acquisition system and capabilities are presented in this section together with details of test material and specimen preparation technique. The resolutions of transducer measurements are also noted.

2.1 Hollow Cylinder Torsional (HCT) shear device

The HCT testing system was custom built by AllpaTech Geotechnical Instruments Inc. of Richmond, BC. A photograph of the device showing the Digital Pressure Volume Controllers (DPVC) and drainage chambers together with a test specimen is shown in Figure 1. This device uses a specimen with an outer diameter of 150mm, inner diameter of 100mm and a height of 300mm. These dimensions are chosen to minimize the stress non-uniformities across the wall under typical test conditions. This device is equipped with high speed (333 kS/s) and high resolution data acquisition system connected to state of the art Electro-Pneumatic Transducers (EPT), Stepper Motor Drives (SMD), and high precision transducers to enable confident and repeatable measurements of loads, volume changes and displacements. Three EPTs enable computerised control of inner pressure, outer pressure and vertical load. The stress and displacement controlled torque loading is applied using two SMDs attached to the HCT device as shown in Figure 2. The torque load arrangement targets the shear stress or shear displacement depending on the test requirement. The target shear stress is applied using a feedback control loop established in the data acquisition program. Three DPVCs and the torque loading arrangement enables the strain path controlled testing. The three DPVCs enable precise control of inner volume, sample volume and the vertical displacement. By controlling inner volume, sample volume, vertical displacement and rotational displacement, a desired strain path can be imposed in a soil sample. The use of DPVCs in soil testing has been pioneered by Menzies (1988), and in strain path testing by Chu et al. (1992).

A total of nine transducers have been used in the HCT apparatus to measure the stress and strain components. Three pressure transducers are used to measure the inner, outer and sample pore pressure with a resolution of 0.05kPa. The vertical and torque loads are measured by a thrust-torque load cell. This load cell has a resolution of approximately 0.05Nm for torque measurements (shear stress in the order of 0.1kPa) and approximately 0.5N for axial load measurements (vertical stress in the order of 0.05kPa). The cross talk between torque and thrust measurements is negligible in the load cell. Also, the vertical load can also be measured by an extra (independent) load cell attached beneath the torque loading system. This load cell has resolution similar to that of the combined load-torque cell (vertical stress in the order of 0.05kPa).

The DPVCs are combined with an actuator and a stepper motor drive. The actuator injects or withdraws the water depending on the applied pulses to the SMD. Physically, each pulse injects (or withdraws) 5.5 x 10^{-4} ml of water. Resolution of the vertical strain control depends on the relative diameter of the axial piston and the DPVC piston. In the current configuration, a volume injection (or

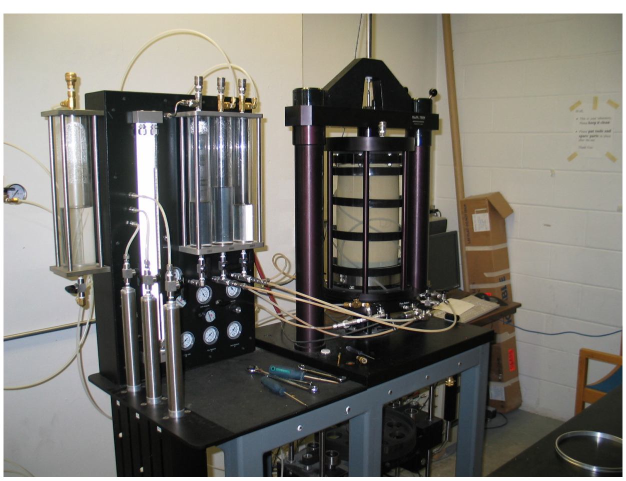

Figure 1. Hollow Cylinder Torsional (HCT) shear device at Carleton University

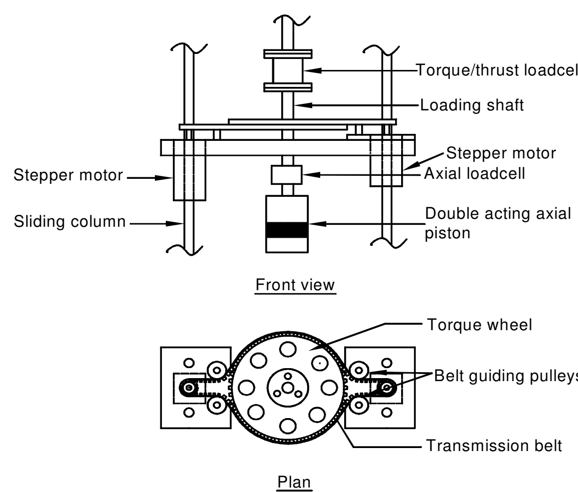

Figure 2. Torsional loading system in the HCT device

withdrawal) of approximately 72ml is required to apply 1mm of vertical displacement to the soil sample (using the DPVC connected to the bottom of the axial double acting piston). This translates to approximately 7.5 x 10^{-6}mm of axial movement per each pulse applied to this DPVC. The resulting axial strain resolution is about 2.5 x 10^{-8}. Each pulse applied to the DPVCs connected to the inner and sample volume causes volumetric strains of 2.5 x 10^{-8} and 2.0 x 10^{-8} respectively. However, in reality, the accuracy of the axial and volumetric strain measurements and the ability to closely follow a prescribed strain path will depend on the accuracy of the timer used to control the hardware. Traditional data acquisition programs which utilize the internal system clock of the personal computer to time events have a timer resolution of about 55ms.

A multithreaded data acquisition program was developed in-house to acquire the data, and control the system. Multiple execution threads to scan transducer readings, save the data to a file, control EPTs and SMDs (one thread per each EPT/SMD) within a single process enable continuous and smooth operation of the control hardware, and proper sampling of the input channels without interruption or delay.

2.2 Testing material

The sand tested was dredged from the Fraser Delta near Abbortsford, British Columbia. Initially, it was wet-sieved through #200 sieve to remove the fine particles and then dry-sieved thorough #20 sieve to remove the coarse particles. The removal of coarse and fine material yields fairly uniform sand with mean diameter 0.30mm, and uniformity coefficient 2.9. Such uniform material facilitates fundamental laboratory studies that require several repeatable, homogeneous specimens. The Fraser Delta sand has been used in several past studies and reported in the literature (Vaid and Thomas, 1994; Vaid and Sivathayalan, 1996; Wijewicreme et al., 2005; Logeswaran and Sivathayalan 2005).

The maximum and minimum void ratios of this batch of Fraser Delta sand determined according to the ASTM test standards (ASTM D4253, D4254) are 0.806 and 0.509. The mineral composition of the sand is similar to the various batches of Fraser Delta sand discussed in the literature. It has been identified that the differences in the gradation and the geographical origin cause fairly significant changes in the maximum and minimum void ratios.

2.3 Specimen preparation technique

Specimens were reconstituted by the water pluviation technique (Vaid and Negussey, 1988), which duplicates the natural deposition process of alluvial/fluvial soil deposits. It has been noted in the literature that water pluviation yields the fabric similar to that of natural sands (Vaid et al., 2001). As a result, the measured laboratory response may be directly applicable to in-situ soils that were deposited in a hydraulic environment. In addition, the very high repeatability of this specimen preparation technique permits the reconstitution of several identical specimens, which is an essential requirement in fundamental experimental studies.

3 TEST RESULTS AND DISCUSSION

The HCT tests presented were carried out on hydrostatically consolidated and fully saturated specimens. A simple triaxial test was simulated as a first check to assess whether the HCT device reproduces the well established triaxial compression behaviour of this sand. Repeatability of the specimen preparation technique and capability of HCT device is further demonstrated by undrained tests on Fraser River sand along different total stress paths. In addition, the sand responses under imposed strain paths are also presented. In these strain path controlled tests, soil specimen was consolidated to an isotropic effective stress of 200 kPa and sheared at a constant rate of shear strain rate. The volume change was controlled using the DVPCs such that the volumetric strain was proportional to the maximum shear strain at all times.

3.1 Monotonic undrained tests

Figure 3 compares the response of Fraser River sand under conventional triaxial compression loading mode measured using the HCT device to that reported in the literature using triaxial device. Triaxial conditions were simulated in the HCT test by maintaining constant inner and outer pressures. Axial deformation was monotonically increased by using a DPVC. Relative densities of the samples at end of the consolidation were essentially similar (20% in triaxial, and 19% in HCT), and the samples were consolidated to 200kPa hydrostatic stress prior to undrained shear. Stress-strain response matches excellently, and the pore pressure responses match fairly closely. Since the triaxial data in the literature did not consider membrane penetration, no membrane penetration corrections were applied in the HCT test. The larger membrane surface area to volume in an HCT device leads to a relatively larger membrane penetration effect in the HCT test compared to triaxial. This can be seen by the marginally smaller excess pore pressure generated in the HCT simulation, and the resulting deviations in the stress path.

Figure 4 illustrates the undrained response of Fraser River sand, but under different total stress paths. The first undrained test was performed using the HCT device to duplicate the conditions in a typical triaxial compression test (under constant total lateral stress). As a result, this test represents shear deformation under increasing mean normal stress. A second test was performed under triaxial compression loading conditions, but the total stress was held constant throughout the deformation. In both tests, essentially the same relative density was achieved at each stage of sample preparation -a demonstration of the good repeatability. The relative density of the sample at end of preparation and at the end of consolidation was 15% and 19% respectively. Such fundamental tests (Figure 3 & 4) were performed using the newly built HCT device to monitor the performance of the device and the data acquisition and control systems. The first test in Figure 4 does not require any feedback control, whereas the stress state in the second test was adjusted in real time using a feedback loop.

These undrained tests along different total stress paths have confirmed that the effective stress path and stress-strain response is identical under undrained loading regardless of the total stress paths for a given initial condition. This concept is a well established in the literature (Henkel 1960; Vaid and Campanella 1974; Bishop and Wesley 1975; Vaid et al 1988; Kuerbis 1989). The ability to reproduce this finding is considered evidence that the feedback control systems in the device are functioning satisfactorily.

The loosest deposited Fraser River sand exhibits strain hardening response during the undrained loading. Such response of water deposited Fraser River sands have been reported in the literature (Vaid and Thomas 1994; Vaid and Sivathayalan 1996, Sivathayalan and Logeswaran, 2007 & 2008). The friction angle measured at compression mode of undrained loading for Fraser Delta is 32° at phase transformation and 35° at maximum obliquity. These observations are consistent with the values reported in earlier studies (Vaid and Thomas 1994; Vaid and Sivathayalan 1996; Eliadorani, 2000).

3.2 Undrained tests at different *b* values

The influence of the intermediate principal stress was studied under undrained condition. A series of tests were conducted at *b* values of 0.0, 0.4, 0.7, and 1.0. The intermediate stress parameter *b* and total mean normal stress were held constant at specific values. In addition the principal stress direction α_σ was held constant along the deposition direction in each of the tests. It can be noted in Figure 5 that the stress-strain response, the pore pressure and the undrained strength were not affected significantly up to *b* values of 0.7. However, the strength is marginally lower at of *b* value of 1.0. This observation is consistent with previous studies reported in the literature (Uthayakumar, 2005). Also, Figure 5 shows the variation of intermediate principal strain ε_2 in the undrained loading. The magnitude of ε_2 changes from negative to positive when *b* value increases from zero to 1.0. Since the strain increment directions are essentially constant, the strain paths are linear in this strain space. It is also noted that the intermediate principal strain is

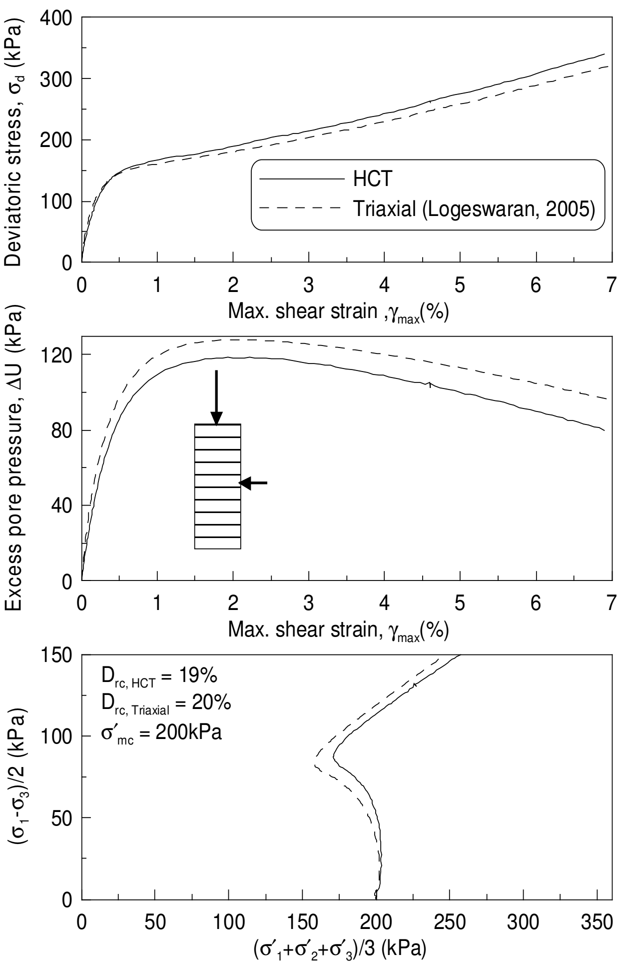

Figure 3. Axisymmetric compression response measured using Triaxial and HCT devices

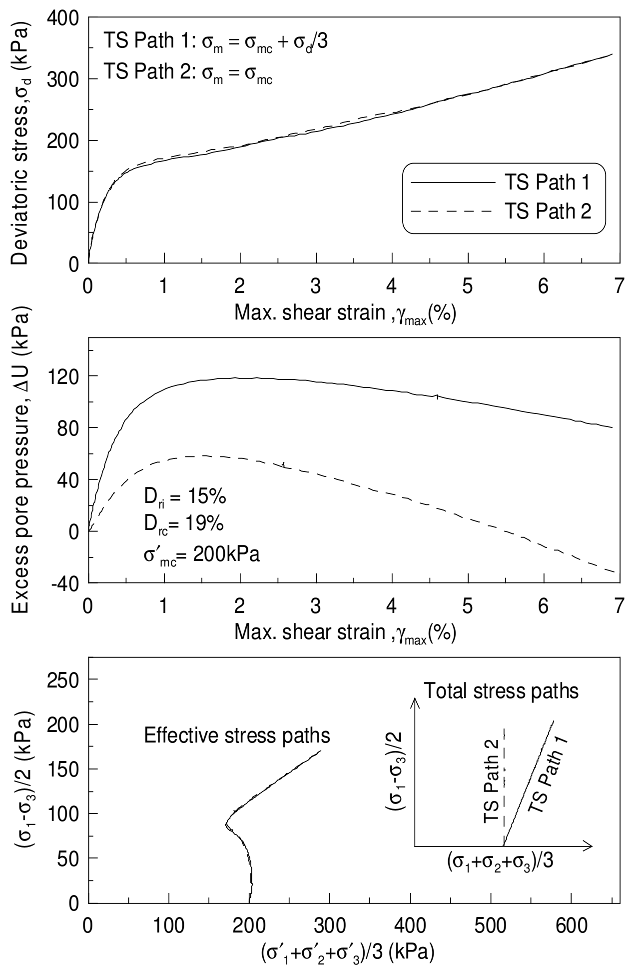

Figure 4. Undrained Response of sand subjected to different total stress paths

approximately close to zero when the *b* value is 0.4. This implies that *b* = 0.4 yields close to plane strain conditions. Previous studies on drained and undrained behaviour have also reported that the *b*=0.4 may closely reflect the plane strain mode of loading (Sayao 1989; Uthayakumar 1995).

3.3 Strain path controlled tests

Linear strain path was imposed on loosest deposited samples consolidated to hydrostatic effective stress of 200kPa. A constant ratio of -0.1 was maintained between volumetric strain and maximum shear strain during shearing. The negative ratio implies expansive volumetric strain (increment of sample volume) during the shear deformation. The DPVC connected to the pore space would inject the appropriate amount of water into the sample depending on the level of maximum shear strain the sample experiences.

As shown in Figure 6, a systematic variation of strength was observed during the strain path controlled loading. The variation of strength during imposition of the strain path is insignificant when *b* value is less than 0.4. The sand exhibits strain hardening behaviour even though the application of expansive volumetric strain (water injection) has made it somewhat softer than the undrained. The strain hardening tendency reduces when *b* value increase. A significant reduction of strength was observed at high value of *b*. The *b* values of 0.8 and 1.0 exhibit marginally strain softening response. The sand exhibits essentially constant shear strength after reaching the peak strength at relatively high *b* values.

The Figure 6 also shows the relationship between intermediate principal strain ε_2 and maximum shear strain γ_{max}. The magnitude of ε_2 changes negative to positive when *b* value increases from zero to 1.0 as noted in undrained loading. The strain path controlled tests also exhibit a linear strain path in strain space. The ε_2 during strain controlled shearing is approximately zero when *b* value is 0.4, as noted in corresponding undrained test. These results suggest that *b* value of 0.4 yields essentially plane strain conditions even in partially drained tests.

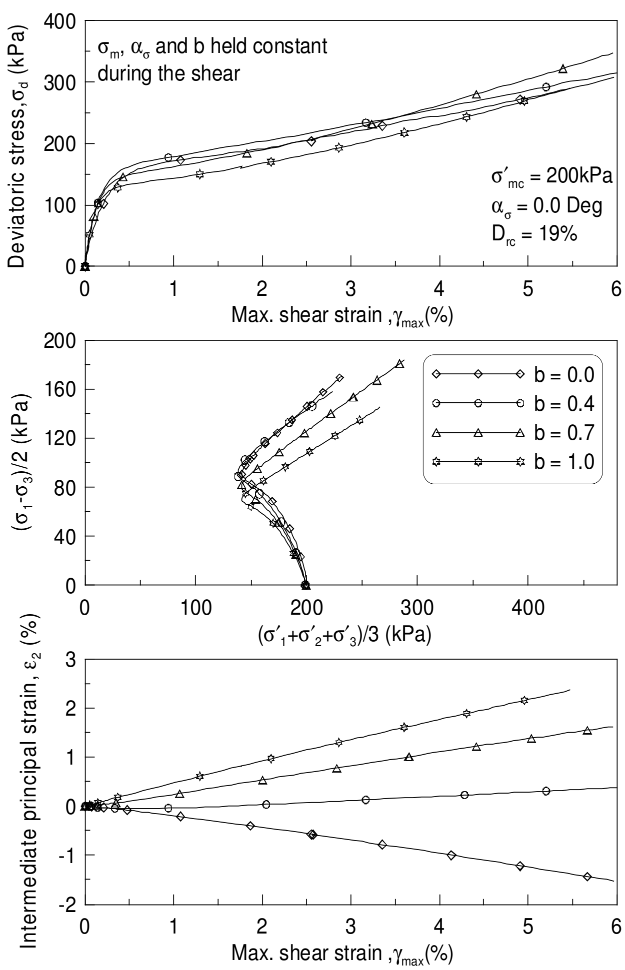

Figure 5. Effect of intermediate principal stress parameter, *b* on the undrained response

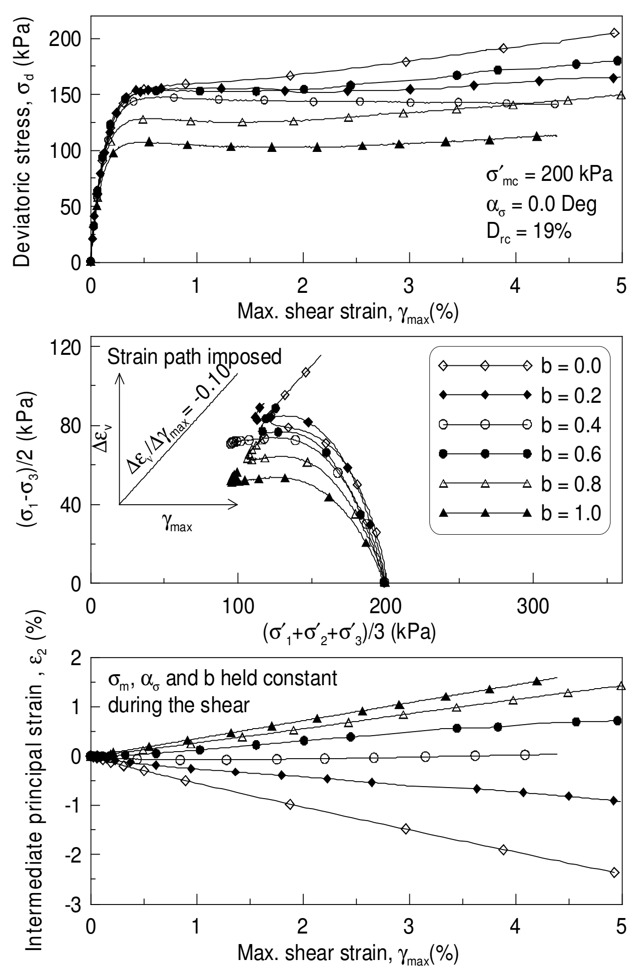

Figure 6. Strain paths controlled loading response of Fraser River sand at different *b* values

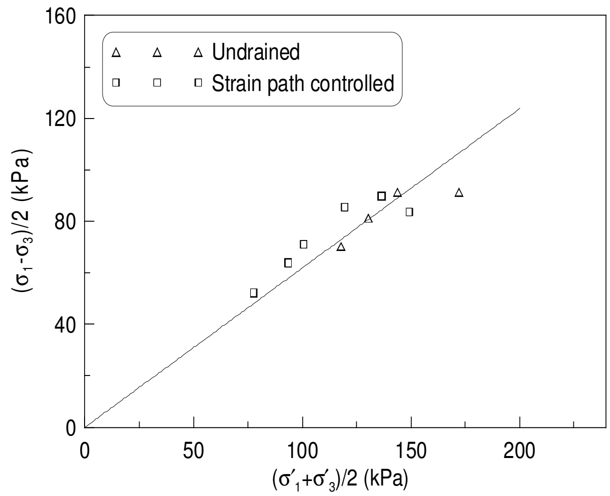

Figure 7. Stress state at maximum pore pressure state

The Figure 7 shows the stress state at maximum pore pressure state at undrained tests and strain path controlled tests at different *b*. It can be noted that the friction angle at peak pore pressure can be treated as unique material property. The peak pore pressure state at undrained test referred as phase transformation. From the previous study, friction angle at phase transformation is independent on the *b* values (Uthayakumar, 2005) and it is also independent on the drainage condition (Sivathayalan and Logeswaran, 2007).

4 SUMMARY AND CONCLUSIONS

A new HCT device has been commissioned at Carleton University. The ability to duplicate triaxial results reported in the literature, and consistent undrained response in tests along different total stress paths indicate the performance of the device is satisfactory. These initial tests provide the confidence required in using a new experimental device.

An experimental study was undertaken to assess effect of intermediate principal stress parameter *b* on behaviour of Fraser Delta sand under strain path controlled loading. The stress-strain response of the sand during the strain path loading systematically changes as *b* value increase from zero to 1.0. The changes are fairly minor for b values ranging from about 0 to 0.5, but softer response was noted at higher values of b. Strain softening could be triggered in sands which are dilative and stable under undrained conditions, if the sand is subjected to very small levels of expansive drainage. Friction angle mobilised at the state of peak pore pressure state appears to be a unique material property.

Current practice, which routinely considers drained and undrained responses only, and presumes that these form the bounds of all possible responses may result in unsafe designs, if the pore pressure distribution in-situ result in expansive volumetric deformation in soil elements in-situ. This represents a critical risk if such materials are evaluated using traditional undrained considerations.

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