Effect of Specimen Preparation Method on the Stress-Strain Behavior of Sand in Plane-Strain Compression Tests

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Effect of Specimen Preparation Method on the Stress-Strain Behavior of Sand in Plane-Strain Compression Tests

ABSTRACT: Experimental results are presented in this paper to study the effect of specimen preparation method on the stress-strain behavior of sand in plane-strain compression tests. The data obtained from K_0 consolidation, drained, undrained and strain path tests conducted on medium loose specimens prepared by the moist-tamping (MT) and the water sedimentation (WS) methods are compared. The test data show that the plane-strain compression behavior of medium loose sand under K_0, drained and strain-path controlled (including undrained) conditions is affected by the specimen preparation method. Under K_0 conditions, the K_0 values obtained from the MT specimens are generally lower than those obtained from the WS specimens. Under drained conditions, more contractive behavior was observed for the MT sand. However, the failure stress ratio (or the failure friction angle) was not affected by the specimen preparation method. The data presented in this paper also illustrate that the compression behavior of medium loose sand in strain-path testing can be affected by the specimen preparation method. However, the differences in the stress-strain behavior will also depend on the strain increment ratio ($d\varepsilon_v/d\varepsilon_1$) imposed on the specimens. In general, different behaviors of the moist-tamped and water-deposited specimens reflect the influence of soil fabrics on the stress-strain behavior of sand.

KEYWORDS: sand, plane-strain, stress-strain behavior, sand fabric, pluviation, moist tamping

Introduction

Several preparation methods of granular soil specimens can be used in soil mechanics laboratories. Moist tamping (MT), water sedimentation (WS) (also known as water pluviation), and air pluviation (AP) are among the most popular techniques. In the MT method, moist granular soil is deposited into a mold in a few layers and each layer is compacted using a small tamper. In the other two preparation methods (i.e., WS and AP), a granular material is poured into the mold, which is either empty (AP method) or partially filled with water (WS method). If necessary, the density of the specimens prepared by pluviation methods can also be increased by vibration or tamping.

It has long been recognized that different preparation methods result in different fabrics of granular soils and, consequently, in different stress-strain characteristics of reconstituted specimens (Oda 1972a, 1972b; Ladd 1974, 1977; Mulilis et al. 1977; Silver et al. 1980; Miura and Toki 1982; Kuo and Frost 1996; Frost and Park 2003; Yamamuro and Wood 2004). Owing to this, a number of experimental studies discussing the various effects of sample preparation methods on the stress-strain behavior of granular soils have been reported in the past. However, the majority of experiments have been conducted under axisymmetric conditions using triaxial cells. Studies investigating the effects of different specimen preparation methods on the stress-strain behavior of sand under more generalized stress conditions, such as plane-strain, are very rare even though several field problems (e.g., slopes, embankments, or retaining walls) cannot be approximated to axisymmetric conditions. This is partially due to the fact that plane-strain devices are not commonly available and plane-strain tests are more complicated to conduct than triaxial tests. As a result, our understanding of the effects of specimen preparation methods on the stress-strain behavior of soil under plane-strain conditions is still very limited.

The main objective of this paper is to study the effects of specimen preparation methods on the stress-strain behavior of sand under plane-strain conditions. Several K_0 consolidated plane-strain compression tests conducted on medium loose sand under various drainage conditions were carried out. The results obtained from the plane-strain tests conducted on medium loose specimens prepared by the moist tamping and the water sedimentation methods are compared. The effects of specimen preparation methods on the measurement of K_0 values and the stress-strain behavior of sand in plane-strain tests are discussed.

Previous Studies

In the past, several investigators have studied the effect of specimen preparation methods on stress-strain behavior of granular soil. Among the first attempts to study the effects of the specimen preparation methods on the behavior of reconstituted sand were those of Ladd (1974, 1977), who observed that the method of specimen preparation could have a significant effect on the cyclic shear strength of sand. Similar observations were made by Mulilis et al. (1977). They reported that different specimen preparation procedures significantly affected the liquefaction characteristics of sand in undrained stress-controlled cyclic triaxial tests. Mulilis et al. (1977) also observed that the MT specimens were more nonuniform than the others. Therefore, in order to improve the uniformity of the MT specimens Ladd (1978) proposed an undercompaction procedure. In this method, the specimen is prepared using a number

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of layers and each layer is compacted to a selected percentage of the required density of the specimen. Mulilis et al. (1978) also verified that the MT specimens prepared by the undercompaction method were more uniform than comparable specimens prepared by the constant compactive effort method proposed by Castro (1969). Vaid and Negussey (1988), on the other hand, promoted the pluviation methods. They reported that pluviated specimens were more uniform than moist-tamped specimens. A detailed study of density variations in sand specimens conducted by Gilbert and Marcuson (1988) has shown, however, that some nonuniform density redistributions within sand specimens are unavoidable, regardless of the preparation method.

The effect of the specimen preparation method on the steady state of granular soils (Poulos 1981) has also been studied. Dennis (1988) observed that the specimen preparation method affected the slope of the steady state line for stress-controlled loading but not for strain-controlled loading. DeGregorio (1990) also reported that the steady state line determined by undrained monotonic triaxial tests was affected by the sample preparation method. In contrast, Hird and Hassona (1990) observed that the specimen preparation technique has no apparent effect on the steady state. Similar observations were made by Verdugo and Ishihara (1996) and Zlatovic and Ishihara (1997). A number of researchers have also reported that moist-tamped specimens were generally more susceptible to liquefaction than water- or air-pluviated specimens (DeGregorio 1990; Hird and Hassona 1990; Vaid et al. 1999; Vaid and Sivathayalan 2000; Chu et al. 2003a; Eliadorani and Vaid 2003). Therefore, as noted by Leong and Chu (2002), the results obtained from tests conducted on specimens prepared by one method should not be generalized to specimens prepared by other methods.

Ishihara (1993), in the 33rd Rankine Lecture, emphasized that apart from the basic requirement to produce homogeneous specimens, a preparation method should also be able to cover a wide density range of reconstituted specimens. He reported that the widest range of void ratio can be achieved using the MT method, allowing both the contractive and dilative behaviors of sand to be studied. Vaid et al. (1999) argued that the MT method does not simulate the fabrics of naturally deposited alluvial and hydraulic fill sands. Therefore, the use of the MT specimens for element testing might be questionable (Vaid et al. 1999; Vaid and Sivathayalan 2000; Eliadorani and Vaid 2003). Chu and Leong (2003), on the other hand, have pointed out that not all practical problems can be simulated by the pluviation method; e.g., a case where moist sand is truck-dumped as a fill material and subsequently submerged as the water table rises. Furthermore, some of the most important concepts describing the general behavior of sand, such as the steady state (Poulos 1981) or the state parameter (Been and Jefferies 1985), have been established based on data obtained from tests conducted on moist-tamped specimens.

More recently, Yoshimine and Koike (2005) reported that homogeneous and uniform sand specimens prepared in the laboratory by either method do not generally resemble natural soil deposits. This is because soil structures commonly observed in nature are layered and stratified rather than homogeneous. Therefore, Yoshimine and Koike (2005) emphasized that establishing a link between different fabrics and structures obtained from various preparation methods in the laboratory and the *in situ* sedimentation and stress histories is very important from a practical point of view. Papadimitriou et al. (2005) reported that such a link could be established by using a plasticity model with the inherent fabric anisotropy scheme, proposed by Dafalias et al. (2004). However, more experimental data on the effects of specimen preparation method on the stress-strain behavior of sand under various stress and drainage conditions are needed to improve further modeling.

To date, studies comparing the effects of different specimen preparation methods on the stress-strain behavior of sand under more generalized stress conditions (e.g. plane-strain, simple shear, or multi-axial) are still very limited. For instance, Finno et al. (1997) studied the effect of the specimen preparation methods in drained plane-strain tests, whereas Vaid et al. (1999), Vaid and Sivathayalan (2000), and Wijewickreme et al. (2005) used undrained direct simple shear tests. They reported that the moist-tamped specimens were more nonuniform and weaker than the water- or air-pluviated specimens. Furthermore, most experimental studies on the effects of specimen preparation methods have been conducted under undrained conditions. However, in most practical problems, a truly undrained condition is exceptional because an *in situ* soil element will normally experience both volume change and excess pore water pressure simultaneously (Chu et al. 1992; Vaid and Eliadorani 2000). This situation can only be simulated in the laboratory by a strain path testing method (Chu and Lo 1991), in which the strain increment ratio, $d\varepsilon_v/d\varepsilon_1$, imposed on a specimen, is controlled. To the authors' knowledge, a comparison of different specimen preparation methods in strain path testing has not yet been reported. Therefore, more experiments need to be conducted under plane-strain or other generalized stress conditions using the strain path testing method.

Test Arrangement

The plane-strain test system developed by Wanatowski and Chu (2006) was used in this study. A prismatic soil specimen 120 mm in height and 60 by 60 mm in cross section was tested. Two 35 mm thick by 74 mm wide by 120 mm high rigid vertical platens were fixed in position by two pairs of horizontal tie rods to impose a plane-strain condition. The lateral stress in this direction (σ_2) was measured by four submersible total pressure transducers. Two transducers were used for each platen, so that the lateral pressures at both the top and the bottom positions of the specimen could be measured and any non-uniform stress distribution could be detected. The total lateral pressure was evaluated as an average value obtained from the four individual transducers. All rigid platens were properly enlarged and lubricated using a free-end technique (Rowe and Barden 1964) to reduce the boundary frictions and to delay the occurrence of non-homogeneous deformations. For the top and base platens, latex disks were used, whereas for the two vertical platens, Teflon® sheets were adopted. A pair of miniature submersible linear variable differential transformers (LVDTs) was used to measure the vertical displacement. An external LVDT was also used to measure the axial strain when the internal LVDTs ran out of travel. A digital hydraulic force actuator was mounted at the bottom of a loading frame to apply axial load. The actuator was controlled by a computer via a digital load/displacement control box. A 10 kN submersible load cell was used to measure the vertical load. The cell pressure was applied through a digital pressure/volume controller (DPVC). Another DPVC was used to control the back pressure from the bottom of the specimen while measuring the volumetric change at the same time. A pore pressure transducer with a capacity of 1000 kPa was also used to record the pore water pressure at the top of the specimen. For details of the plane-strain apparatus, see Wanatowski and Chu (2006).

Material Tested

The granular soil tested in this study was a marine dredged silica sand, the so-called Changi sand, used for the Changi land reclamation project in Singapore (Leong et al. 2000). The Changi sand has the specific gravity (G_s) of 2.60, the mean grain size (D_{50}) of 0.30 mm, the coefficient of uniformity (C_u) of 2.0, and the coefficient of curvature (C_c) of 0.8. The fines' content is approximately 0.4 %. According to the Unified Soil Classification System (ASTM D2487-06) it is medium grained, poorly graded, clean sand. The individual particles of the sand are mainly subangular in shape. The minimum and maximum void ratios were 0.533 and 0.916, respectively. The minimum void ratio ($e_{\min}$) was determined according to ASTM D4253-00 and the maximum void ratio ($e_{\max}$) according to ASTM D4254-00. Since the Changi sand is dredged from the seabed, it contains shells of various sizes ranging from 0.2 to 10 mm. The shell content of the Changi sand is approximately 12 %.

Specimen Preparation and Testing Procedures

Laboratory reconstituted specimens were used in this study. Two different specimen preparation methods, the water sedimentation (WS) and the moist tamping (MT) were employed. A four-part split mold was used for the preparation of all the specimens. A 0.4 mm thick latex membrane was fitted into the mold. A vacuum pressure of 10 kPa was used to achieve a tight fit between the mold and the membrane. The top cap and the bottom pedestal were designed in such way that a transition from a square to a circular cross section was permitted so cylindrical-shaped membrane could be used. In the WS method, sand was pluviated into the mold which was half-filled with de-aired water. Deposition of sand was done by moving the tip of the funnel in a circular motion 1–2 cm above the water surface. In the MT method, the oven-dried sand was first mixed with 5 % of de-aired water. After mixing, the moist sand was deposited into the mold in five layers and each layer was compacted using a small tamper. The number of blows applied for each layer was carefully controlled. To achieve a greater uniformity of specimens, the undercompaction method, proposed by Ladd (1978), was used. For each layer, the compactive effort was increased towards the top with the undercompaction ratio of 2.5 %. For saturation, the specimen was flushed with de-aired water from the bottom to the top for 60 min under a water head of about 0.5 m. After that a back pressure of 400 kPa was applied. The Skempton's pore water pressure parameter (B-value) greater than 0.96 was obtained for all the specimens. A liquid rubber technique (Lo et al. 1989) was adopted to reduce the bedding and membrane penetration errors. For more details of the sample preparation procedures see Wanatowski and Chu (2006).

In order to compare different specimen preparation methods, several pairs of plane-strain compression tests were conducted. Each pair of tests was conducted on two duplicated specimens by following the same stress or strain path. However, one specimen was prepared by the MT method and the other by the WS method. All the specimens were consolidated from an initial isotropic stress state of 20 kPa to the required stress state along the K_0 path. The K_0 condition was imposed by regulating the volume change of the specimen in accordance with the axial strain to maintain $d\varepsilon_v/d\varepsilon_1 = 1$, a method proposed by Lo and Chu (1991). Shearing was carried out using either a stress or a strain path control. A drained test was conducted using a stress path with $d\sigma_3' = 0$. All other tests were carried out using a strain path method with $d\varepsilon_v/d\varepsilon_1 = \text{const}$ (Chu and Lo 1991). In this method, an undrained test is only a special case of general drainage conditions with $d\varepsilon_v/d\varepsilon_1 = 0$. All the other drainage conditions can be simulated by $d\varepsilon_v/d\varepsilon_1 \neq 0$. When $d\varepsilon_v/d\varepsilon_1 > 0$, the soil element will compress and water will flow out of the specimen with axial deformation. Hence, the pore water pressure inside the specimen will reduce. On the other hand, when $d\varepsilon_v/d\varepsilon_1 < 0$, the specimen will dilate and water will flow into the specimen. If the specimen dilates more than the sand would under a drained condition a positive pore water pressure will develop. Furthermore, the more negative the $d\varepsilon_v/d\varepsilon_1$ imposed on the specimen, the more positive excess pore water pressure will be generated.

It needs to be pointed out that the pore water pressure developed in an undrained strain path ($d\varepsilon_v/d\varepsilon_1 = 0$) is not the highest, as often assumed, but it is in between that for a dilative ($d\varepsilon_v/d\varepsilon_1 < 0$) strain path and a compressive ($d\varepsilon_v/d\varepsilon_1 > 0$) strain path. It means that an undrained condition is not the most dangerous drainage situation, as is often assumed. For instance, dense sand that exhibits strain hardening behavior under an undrained condition will soften in similar way to loose sand when it is subjected to an adequate dilative strain path, as observed by Chu et al. (1992), Vaid and Eliadorani (1998), and Lancelot et al. (2004). As such, only undrained ($d\varepsilon_v/d\varepsilon_1 = 0$) and dilative ($d\varepsilon_v/d\varepsilon_1 = -0.2$ and -0.6) strain path tests were carried out in the present study.

All the plane-strain tests were carried out under a deformation-controlled loading mode at a constant rate of 0.05 mm/min. It should be pointed out that the back pressure level at the end of K_0 consolidation was taken as the datum and the decrease in pore water pressure was noted as negative. A summary of the plane-strain tests conducted is given in Table 1.

In this study, the deviatoric stress q and the mean effective stress p' are defined as

$$q = \frac{1}{\sqrt{2}}[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]^{1/2} \tag{1}$$

$$p' = \frac{1}{3}(\sigma_1' + \sigma_2' + \sigma_3') \tag{2}$$

where σ_1, σ_2, and σ_3 are major, intermediate, and minor principal stresses, respectively; the prime refers to effective stress.

Repeatability of Test Results

Confidence in any experimental investigation is highly dependent on the consistency and repeatability of test results. The duplication of the testing procedures described previously, ensured that reproduction of specimens was achieved in all the tests. Figure 1 presents the results obtained from two CK_0D plane-strain tests conducted in the same way on two medium dense specimens. The specimens were prepared by the WS method and K_0 consolidated to a mean effective stress $p_c' = 200$ kPa. Void ratios at the end of K_0 consolidation were $e_c = 0.686$ and $e_c = 0.681$, respectively. Both specimens were brought to the failure state by a $d\sigma_3' = 0$ stress path. Two unloading-reloading cycles were imposed on each specimen. It can be seen from Fig. 1 that a good consistency in stress-strain behavior [Fig. 1(a)] and volume change [Fig. 1(b)] was obtained. Similar repeatability in test results was obtained for all the other plane-strain specimens (Wanatowski 2005)

As mentioned earlier, the total intermediate stress (σ_2) in all the plane-strain tests was calculated as an average value obtained from

TABLE 1—*Summary of plane-strain tests conducted.*

			Initial State[a]		K_0 consolidated State						Peak State					
Test	Preparation Method	Type of Test	e_0	Dr_0, %	e_c	Dr_c, %	p'_c, kPa	q_c, kPa	ε_{1c}, %	K_0	e_p	Dr_p, %	p'_p, kPa	q_p, kPa	ε_{1p}	Figure Number
CK_0D13	WS	drained ($d\sigma'_3=0$)	0.695	57.7	0.686	60.1	200.5	223.7	0.59	0.36	0.710	53.8	439.0	649.1	3.11	1 and 4
CK_0D15	WS		0.689	59.3	0.681	61.4	199.4	217.6	0.52	0.34	0.703	55.6	435.1	643.5	3.13	1 and 4
CK_0D08	MT		0.983	−17.5	0.914	0.5	202.0	166.0	3.41	0.47	0.869	12.3	305.8	357.9	11.79	2 and 4
CK_0Dws	WS		0.738	46.5	0.725	49.9	200.2	186.4	0.73	0.37	0.735	47.3	372.8	526.4	5.06	4 and 6
CK_0Dmt	MT		0.741	45.7	0.722	50.7	201.6	219.5	1.08	0.25	0.718	51.7	316.8	443.6	2.65	4 and 6
CK_0Uws	WS	Undrained strain path	0.761	40.5	0.734	47.5	199.2	205.0	0.85	0.34	0.734	47.5	486.4	660.7	5.44	4 and 7
CK_0Umt	MT	($d\varepsilon_v/d\varepsilon_1=0$)	0.772	37.6	0.739	46.2	196.8	229.6	1.14	0.28	0.739	46.2	386.2	522.0	3.02	4 and 7
$CK_0SP-0.2ws$	WS	Dilative strain path	0.746	44.4	0.730	48.6	202.7	191.9	0.91	0.39	0.745	44.6	282.4	385.2	4.02	4 and 8
$CK_0SP-0.2mt$	MT	($d\varepsilon_v/d\varepsilon_1=-0.2$)	0.755	42.0	0.735	47.3	200.0	210.4	1.17	0.26	0.736	47.0	195.4	239.4	0.16	4 and 8
$CK_0SP-0.6ws$	WS	Dilative strain path	0.764	39.7	0.745	44.6	200.7	200.0	1.10	0.38	0.746	44.4	200.4	220.5	0.18	4 and 9
$CK_0SP-0.6mt$	MT	($d\varepsilon_v/d\varepsilon_1=-0.6$)	0.759	41.0	0.738	46.5	203.6	216.3	1.19	0.25	0.740	46.0	197.2	231.4	0.05	4 and 9
K_0MT1	MT	K_0 consolidation	0.765	39.4	0.750	43.3	200.2	232.7	0.86	0.25	—	—	—	—	—	4 and 5
K_0MT7	MT	($d\varepsilon_v/d\varepsilon_1=1.0$)	0.978	−16.2	0.915	0.3	198.0	157.2	3.29	0.48	—	—	—	—	—	3 and 4
K_0WS4	WS		0.768	38.6	0.756	41.8	202.9	203.5	0.93	0.39	—	—	—	—	—	4 and 5

[a] $\sigma_{30}=420$ kPa, $u_0=400$ kPa, $p'_0=20$ kPa, $q_0=0$ kPa

four individual pressure transducers installed in the two vertical platens. Therefore, it was essential to ensure that σ_2 values measured by the four individual transducers were consistent and reliable. Typical σ_2 versus ε_1 curves obtained from a drained test on a very loose specimen prepared by the MT method are shown in Fig. 2. It can be seen that all the four curves are close to each other during the entire test. Similar observations were made from all the other plane-strain tests presented in this paper. However, it should be pointed out that for medium loose or medium dense specimens, the lateral stress distributions become highly non-uniform in the post-peak region and the four local values of σ_2 will diverge. Furthermore, the point where the σ_2-ε_1 curves start to diverge coincides with the point where shear bands occur, as discussed by Wanatowski and Chu (2006). Therefore, it can be assumed that the stresses and strains within the plane-strain specimens are essentially uniform before the shear band formation.

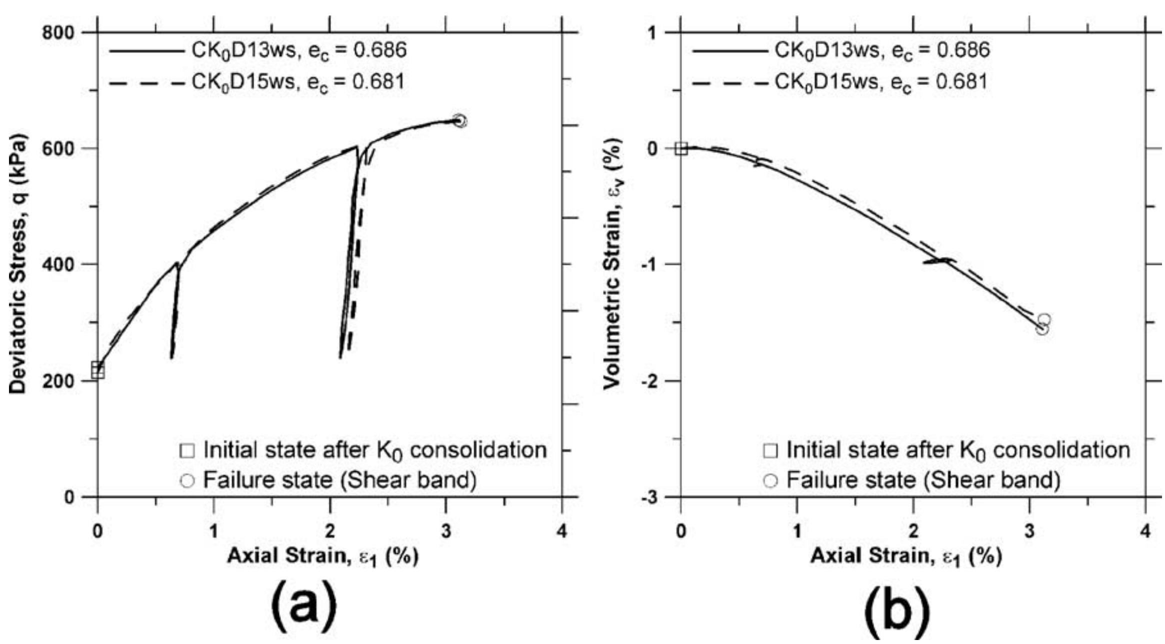

FIG. 1—*Repeatability of drained tests under plane-strain condition: (a) stress-strain curves; (b) volumetric strain curves.*

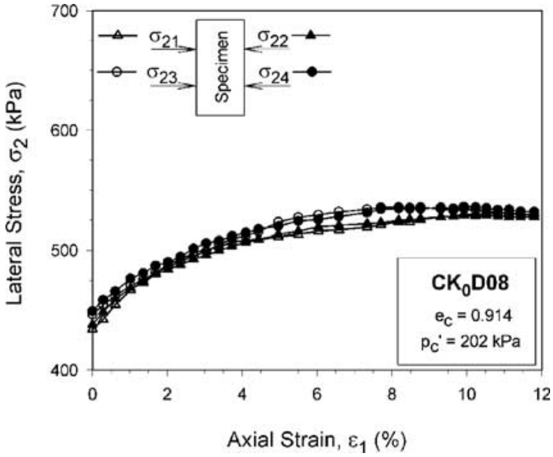

FIG. 2—*The σ_2 versus ε_1 curves obtained for a very loose specimen prepared by the moist tamping method.*

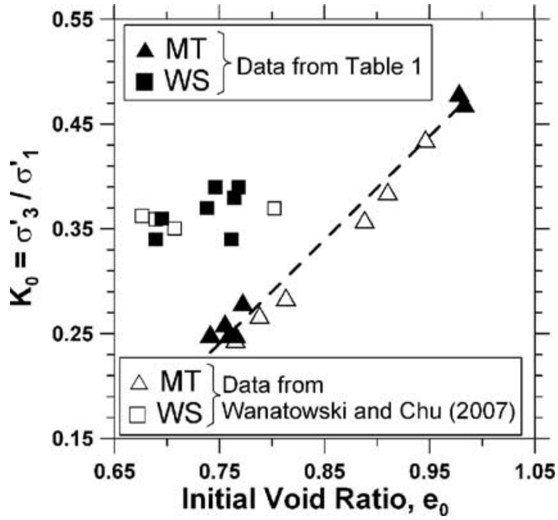

FIG. 4—*K_0 versus e_0 plot determined for Changi sand.*

Experimental Results

K_0 Consolidation

The K_0 behavior of Changi sand in strain path tests conducted in a triaxial cell and a plane-strain apparatus has already been discussed in detail by Chu and Gan (2004) and Wanatowski and Chu (2007). Therefore, only the most important findings will be reported in this paper.

Firstly, the σ_2' and σ_3' versus axial strain curves obtained from a K_0 consolidation test conducted on the MT sand in a plane-strain apparatus are compared in Fig. 3. It can be seen that the two curves are almost identical; that is, $\sigma_2'=\sigma_3'$ is obtained under the $d\varepsilon_2 = d\varepsilon_3 = 0$ condition. Similar observations were made from all the other K_0 consolidation tests conducted on Changi sand in the plane-strain apparatus. This suggests that there is no strong anisotropy in the σ_2' and σ_3' directions; that is, the specimen is essentially cross-anisotropic. Furthermore, this also serves as a verification of the reliability of the plane-strain apparatus used in this study.

Secondly, the K_0 values for Changi sand obtained from this study and the study by Wanatowski and Chu (2007) carried out in a triaxial cell and a plane-strain apparatus are compared in Fig. 4. All the K_0 values were calculated at the end of consolidation stage that is at a mean effective stress of 200 kPa. It can be seen from Fig. 4 that within the range of void ratio tested the K_0 values of the WS specimens fall within a narrow range. In other words, the K_0 values obtained from the tests on WS specimens show little dependence on the initial void ratio. On the other hand, the K_0 values obtained from the tests on MT specimens form a relationship with the initial void ratio. It can be observed from Fig. 4 that the looser the MT specimen, the higher the K_0 value. Chu and Gan (2004) and Wanatowski and Chu (2007) have also reported that K_0 values obtained from the tests on Changi sand do not agree well with Jaky's equation (Jaky 1944) expressed as

$$K_0 = 1 - \sin \phi' \tag{3}$$

where ϕ' is the peak effective friction angle of soil.

It can also be seen from Fig. 4 that the K_0 values obtained from tests on the MT specimens are different from those on the WS specimens at the same void ratio. The K_0 values obtained from the MT specimens are generally lower than those from the WS specimens. This is illustrated further in Fig. 5(*a*) where the K_0-ε_1 curves obtained from two tests conducted on WS and MT specimens with similar void ratios are compared. It can be observed from Fig. 5(*a*) that the higher K_0 value and the lower axial strain were measured during K_0 consolidation of the MT sand. It should also be noted that a K_0 test on sand has to be started from an initial isotropic stress state of 20 kPa. This is because the K_0 consolidation test on saturated sand cannot be commenced from a free stress state. Therefore, there is an initial transition from the isotropic state to the K_0 state. However, this transition only affects the K_0 value at the initial period. The K_0 value approaches more or less a constant value after axial strain exceeds 0.4–0.5 %, which corresponds to the mean effective stress of 80–100 kPa, as shown in Fig. 5(*a*). This observation is consistent with what has been established in a triaxial cell by Lo and Chu (1991) and Chu and Gan (2004). They have reported that an initial isotropic stress does not affect the resulting K_0 path when the consolidation stress is beyond four times the initial stress.

The effective stress paths obtained from the two K_0 tests are plotted in Fig. 5(*b*). It can be seen that the effective K_0 paths obtained from the WS and MT specimens are different. This shows that the specimen preparation method affects not only the K_0 value

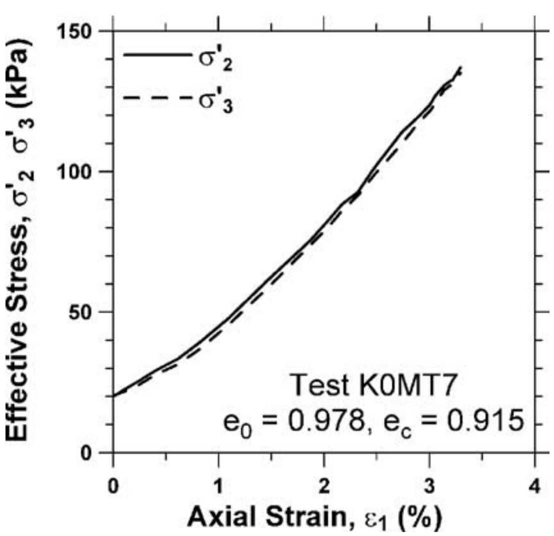

FIG. 3—*The lateral stress response obtained from a K_0 consolidation test.*

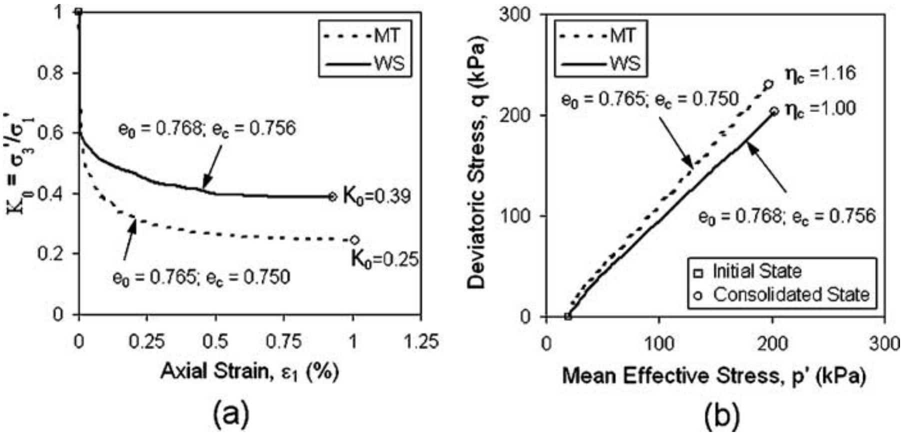

FIG. 5—*Comparison of K_0 consolidation tests conducted on WS and MT specimens.*

of sand but also the effective stress ratio η_c determined at the end of K_0 consolidation. As shown in Fig. 5(*b*), the higher effective stress ratio η_c was obtained for the MT sand. Similar behavior was observed from all the other K_0 consolidation tests conducted in the plane-strain apparatus. As a result, all the plane-strain tests discussed in the following sections are affected by the K_0 consolidation stage; i.e., each K_0 consolidated plane-strain compression test conducted on the MT sand had to be commenced from the higher η_c compared to that conducted on the WS sand with comparable void ratio.

Drained Behavior

The results of two CK_0D tests conducted on medium loose specimens with comparable void ratios are presented in Fig. 6. Both specimens were K_0 consolidated to a mean effective stress p_c' $=200$ kPa and then sheared under drained conditions with σ_3' maintained constant. The stress-strain curves obtained from the two tests are compared in Fig. 6(*a*). It can be observed that the two stress-strain curves are similar. In both tests, the deviatoric stress firstly reached a peak, and then reduced gradually to an ultimate value. However, the peak deviatoric stress obtained from the MT specimen ($q_p=444$ kPa) was lower than that from the WS specimen ($q_p=526$ kPa). It can also be seen from Fig. 6(*a*) that the peak deviatoric stress of the MT specimen was reached at an axial strain of 2.6 %, which is much lower than that of the WS specimen (5.1 %).

The effective stress paths obtained from the two tests are compared in Fig. 6(*b*). The two stress paths are slightly different due to different deviatoric stresses at the end of K_0 consolidation (q_c $=184$ kPa for the WS sand and $q_c=219$ kPa for the MT sand). This is because the effective stress ratio at the end of K_0 consolidation, η_c, obtained from the MT specimen is higher than that from the WS specimen, as explained earlier. Nevertheless, the same failure line with the gradient $\eta_f=1.41$ was obtained from the two tests. It should be noted that the peak (i.e., the failure) was accompanied by a shear band formation in both tests. The ε_v-ε_1 curves of the two tests are compared in Fig. 6(*c*). The two ε_v-ε_1 curves are similar, showing an initial volumetric contraction and a subsequent volumetric dilation. However, the MT specimen behaves more contractively compared to the WS specimen. As a result, the volumetric strains measured at the end of two tests are different. The dilatancy behaviors of the two specimens are compared in Fig. 6(*d*) where the strain increment ratio ($d\varepsilon_v/d\varepsilon_1$) versus axial strain ($\varepsilon_1$) curves are plotted. In both tests, a contractive behavior (i.e., $d\varepsilon_v/d\varepsilon_1>0$) was observed until the characteristic state (Luong 1980) was reached. After that a dilative behavior (i.e., $d\varepsilon_v/d\varepsilon_1<0$) was obtained in the two tests. The characteristic state is defined as the transition point from compression to dilation in a drained test. It can be determined from the ε_v-ε_1 curve as the point where the tangent is horizontal (Luong 1980). As shown in Fig. 6(*d*), the maximum rate of dilatancy was measured at the failure state of each test. As a result, the minimum (i.e., failure) strain increment ratios of $(d\varepsilon_v/d\varepsilon_1)_f$ $=-0.12$ and $(d\varepsilon_v/d\varepsilon_1)_f=-0.25$ were obtained from the MT and the WS specimens, respectively. It can also be observed from Fig. 6(*d*) that the dilatancy rate slowed down in the post-peak regions reaching a constant value at the end of each test.

The *b*-value $[b=(\sigma_2-\sigma_3)/(\sigma_1-\sigma_3)]$ versus ε_1 curves are presented in Fig. 6(*e*). It can be seen that similar *b*-values were measured in the two drained tests. The failure *b*-values of 0.27 and 0.28 were obtained from the MT and the WS specimens, respectively.

Undrained Behavior in Strain Path Testing ($d\varepsilon_V/d\varepsilon_1=0$)

The results of two undrained tests, CK_0Uws and CK_0Umt, conducted on medium loose sand using a strain path method (Chu and Lo 1991) are compared in Fig. 7. The two specimens were K_0 consolidated to $p_c'=200$ kPa and then sheared undrained by maintaining the $d\varepsilon_v/d\varepsilon_1$ ratio at zero. Test CK_0Uws was conducted on the WS specimen and test CK_0Umt on the MT specimen.

The stress-strain curves of the two tests are plotted in Fig. 7(*a*). Similar stress-strain behaviors can be observed in the two tests. It can be seen from Fig. 7(*a*) that after the stress-strain curves reached a peak, strain softening occurred in both tests. The peak states determined from the two tests were accompanied by the formation of shear bands. However, the peak values in the two tests are different and the axial strains at the peak are also different. The higher peak deviatoric stress and the higher axial strain were measured from the WS test, as shown in Fig. 7(*a*). This is consistent with the stress-strain behavior of medium loose sand under drained conditions, shown in Fig. 6(*a*).

The effective stress paths obtained from the two tests are compared in Fig. 7(*b*). Although the stress paths in the two tests are quite similar, they end up at different stress states due to much

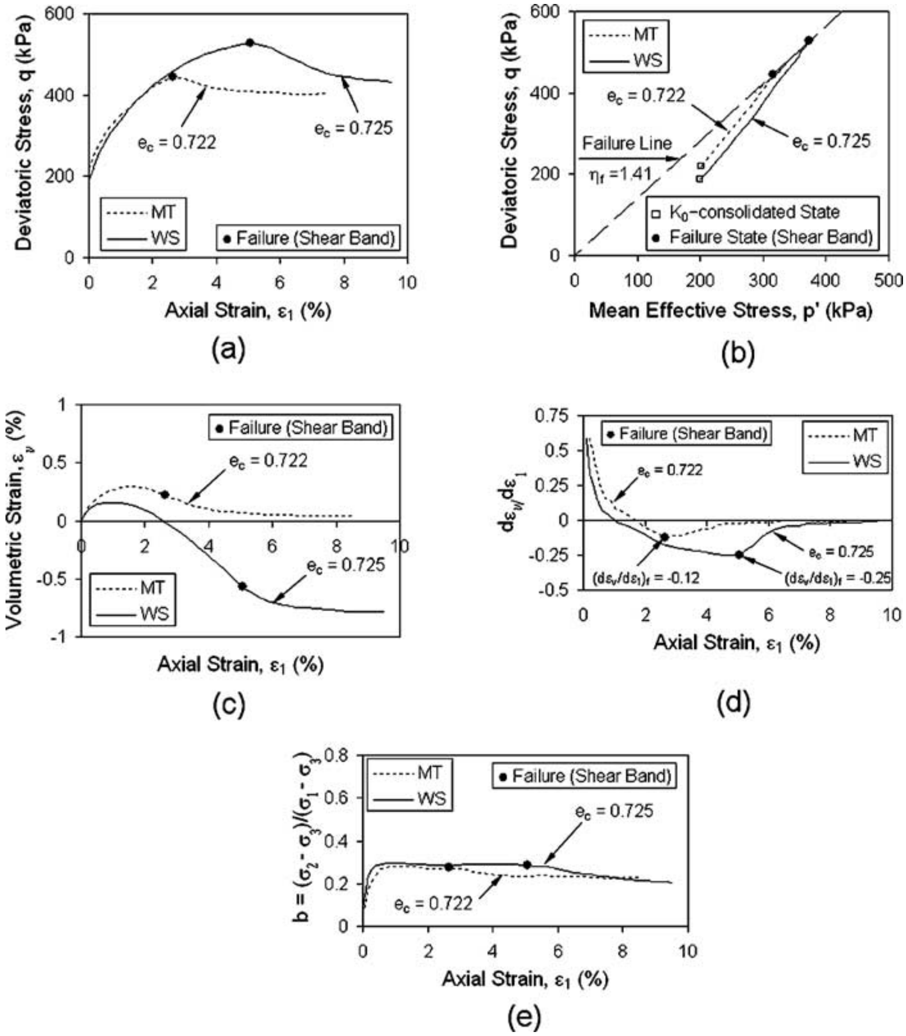

FIG. 6—*Comparison of drained tests conducted on WS and MT specimens.*

lower peak deviatoric stress of the MT specimen. Nevertheless, it can be seen that the two effective stress paths approach asymptotically to a straight line. This line has been called the constant stress ratio line (CSRL) by Chu et al. (2003b). The gradient of this line is $\eta_{asy}=1.37$. This so-called asymptotic behavior (Gudehus et al. 1977) has also been observed for other soils under axisymmetric conditions (Zhang and Garga 1997; Chu et al. 2003b), plane-strain conditions (Topolnicki et al. 1990) and three-dimensional conditions (Chu and Lo 1994).

The excess pore water pressure versus axial strain curves obtained from the two tests are shown in Fig. 7(*c*). Similar pre-peak behaviors were observed for both, the MT and the WS, specimens. However, a more negative excess pore water pressure was generated at the peak point of the WS specimen. As shown in Fig. 7(*c*), the Δu-ε_1 curves of the two specimens are similar in the post-peak region. The excess pore water pressures in the two tests ceased soon after the peak accompanied by shear bands was reached.

The *b*-value versus ε_1 curves of the two tests are plotted in Fig. 7(*d*). It can be observed from Fig. 7(*d*) that both curves are very similar. The peak *b*-values of 0.26 and 0.22 were measured from the MT and the WS specimens, respectively.

Dilative Behavior in Strain Path Testing ($d\varepsilon_v/d\varepsilon_1 < 0$)

The results of two dilative strain path tests with $d\varepsilon_v/d\varepsilon_1=-0.2$ imposed on medium loose specimens reconstituted to comparable void ratios by two different preparation methods are shown in Fig. 8. Both specimens were K_0 consolidated to $p'_c=200$ kPa and then sheared with $d\varepsilon_v/d\varepsilon_1=-0.2$ maintained constant. This means that water was controlled to flow into the specimen and the soil was forced to dilate at a constant rate during the entire shearing stage.

The stress-strain curves obtained from the two tests are compared in Fig. 8(*a*). It can be seen that two different types of behavior were obtained from the MT and the WS specimens even though the same strain increment ratio was imposed on both specimens. As shown in Fig. 8(*a*), strain softening occurred in the MT test, whereas strain hardening occurred in the WS test. In other words, contractive behavior was obtained from the MT specimen, whereas dilative behavior was obtained from the WS specimen. It can be seen from Fig. 8(*a*) that although the WS sand exhibited a strain hardening behavior, strain softening has also occurred in the WS test after the deviatoric stress reached the peak. However, it should be pointed out that the strain softening observed in the WS test is different from that in the MT test. First, the peak of the WS speci-

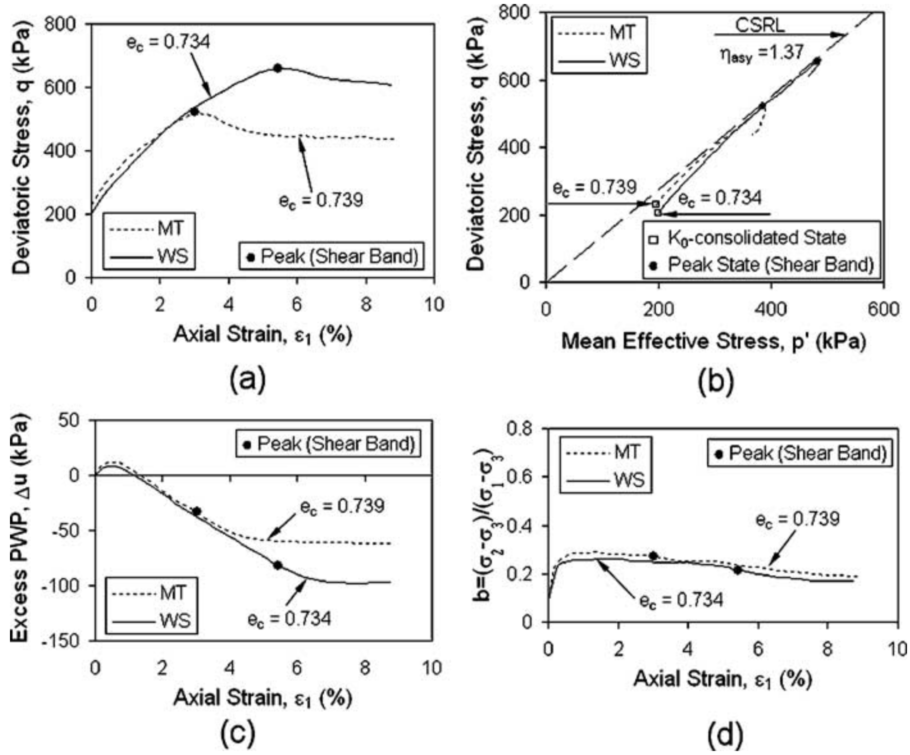

FIG. 7—*Comparison of undrained ($d\varepsilon_v/d\varepsilon_1=0$) strain path tests conducted on WS and MT specimens.*

men was reached at $\varepsilon_1=4.02$ %, whereas the peak point of the MT specimen was reached at $\varepsilon_1=0.16$ %. Second, in the WS test, the peak was accompanied by the shear band formation. However, in the MT test, no shear band has occurred. Therefore, the strain softening observed in the strain path test on the WS sand is due to the development of shear band rather than an element soil behavior. In

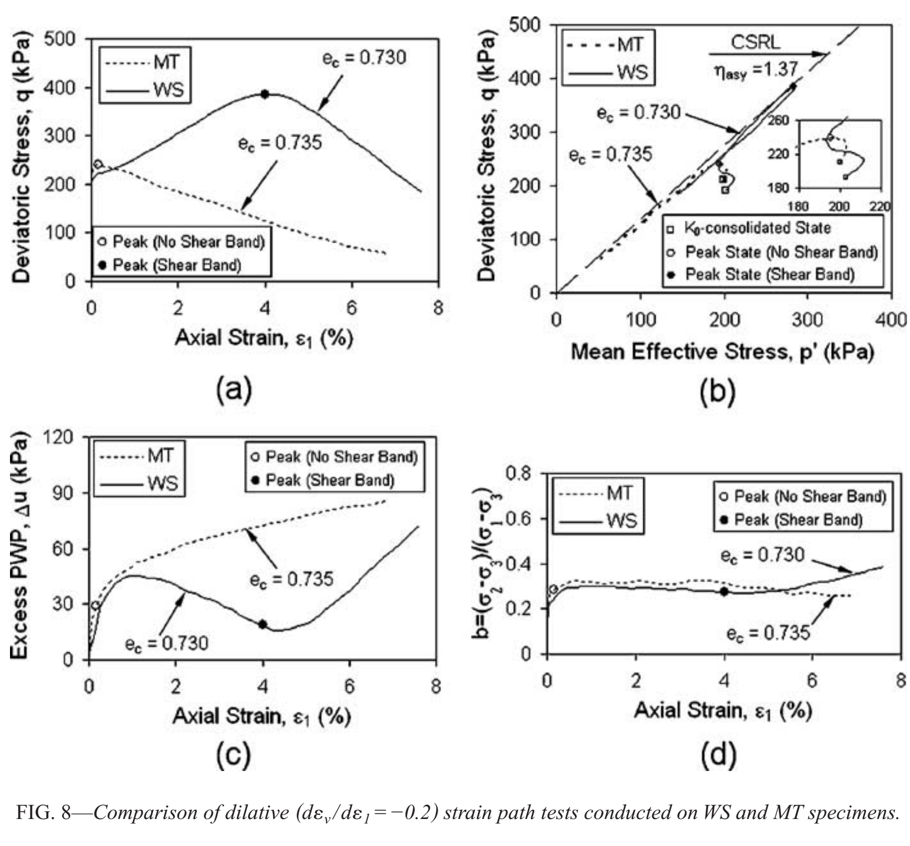

FIG. 8—*Comparison of dilative ($d\varepsilon_v/d\varepsilon_1=-0.2$) strain path tests conducted on WS and MT specimens.*

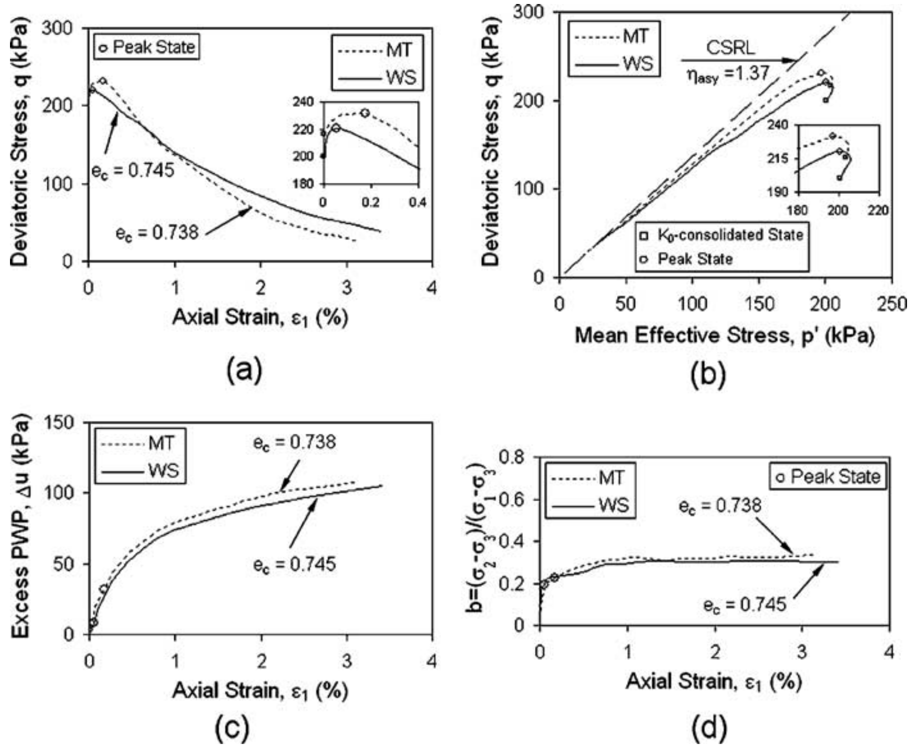

FIG. 9—*Comparison of dilative ($\varepsilon_v/\varepsilon_1=-0.6$) strain path tests conducted on WS and MT specimens.*

other terms, a material softening was observed for the MT sand, whereas a banding softening (Chu et al. 1996, Wang and Lade 2001) was observed for the WS sand.

The effect of the specimen preparation method on the occurrence of strain softening is further demonstrated in Fig. 8(*b*) where the effective stress paths of the MT and WS tests are compared. As shown in Fig. 8(*b*), the effective stress paths traced by the two $d\varepsilon_v/d\varepsilon_1=-0.2$ tests are different. Strain softening is observed in the MT sand but not in the WS sand, as discussed earlier. However, both effective stress paths approach asymptotically to the same constant stress ratio line with the gradient $\eta_{asy}=1.37$. This gradient is consistent with that obtained from undrained tests ($d\varepsilon_v/d\varepsilon_1=0$), shown in Fig. 7(*b*).

The Δu-ε_1 curves obtained from the two tests are shown in Fig. 8(*c*). It can be seen that two different curves were measured in the two tests. The excess pore pressure increased continuously throughout the entire MT test indicating contractive behavior of moist-tamped sand. On the other hand, the pore water pressure in the WS specimen increased initially leading to limited strain softening behavior [see Fig. 8(*b*)], and then reduced gradually, leading to strain hardening behavior [see Fig. 8(*a*)]. Finally, after the shear band development, the excess pore water pressure in the WS test increased again [Fig. 8(*c*)] and the banding type of strain softening behavior was observed, as shown in Figs. 8(*a*) and 8(*b*). It can also be seen in Fig. 8(*c*) that the excess pore water pressure developed in the MT specimen was higher than that developed in the WS specimen.

The *b*-value versus axial strain curves obtained from the $d\varepsilon_v/d\varepsilon_1=-0.2$ path tests are shown in Fig. 8(*d*). The same peak *b*-value of 0.27 was obtained from each specimen. However, the *b*-value measured for the MT sand was almost constant throughout the entire test whereas the *b*-value measured for the WS sand started to increase soon after the shear band had developed [Fig. 8(*d*)].

The results of another pair of dilative strain path tests with $d\varepsilon_v/d\varepsilon_1=-0.6$ imposed on the medium loose specimens prepared to comparable void ratios by two different reconstitution methods are shown in Fig. 9. Both specimens were firstly K_0 consolidated to $p_c'=200$ kPa and then sheared with $d\varepsilon_v/d\varepsilon_1=-0.6$ maintained constant.

The stress-strain curves and the effective stress paths obtained from the two tests are presented in Figs. 9(*a*) and 9(*b*), respectively. It can be observed that the trends of the q-ε_1 and q-p' curves are very similar. The peak deviatoric stress, obtained at a very low axial strain, was followed by strain softening behavior in each test, as shown in Figs. 9(*a*) and 9(*b*). However, a slightly lower value of peak deviatoric stress was yielded in the WS test. This is because the effective stress ratio at the end of K_0 consolidation, η_c, obtained from the MT specimen was higher than that obtained from the WS specimen, as explained earlier. Nevertheless, the same CSRL with a slope $\eta_{asy}=1.37$ was approached by the two effective stress paths, as shown in Fig. 9(*b*). The Δu-ε_1 and b-ε_1 curves for the two tests are plotted in Figs. 9(*c*) and 9(*d*), respectively. It can be seen that the excess pore water pressures and the *b*-values obtained from the two tests are very similar.

Discussion

It is well known that different specimen preparation methods can result in different stress-strain behaviors of the same sand (Ladd 1974; Vaid et al. 1999; Frost and Park 2003; Chu and Gan 2004; Yamamuro and Wood 2004). However, the experimental results discussed in the literature are generally limited to triaxial tests on loose sand under undrained conditions. This is understandable as the static liquefaction of sand observed in such tests can lead to very dramatic and devastating flow failures of loose granular slopes (e.g., Casagrande 1965; Chu et al. 2003b; Olson and Stark 2003). It has been established that loose moist-tamped specimens subjected to undrained loading are generally more susceptible to liquefaction than water- or air-pluviated specimens (DeGregorio 1990; Vaid et al. 1999; Vaid and Sivathayalan 2000; Chu et al. 2003a; Eliadorani and Vaid 2003).

The data presented in this paper further show that the plane-strain compression behavior of medium loose sand under K_0, undrained, and drained conditions is affected by the specimen preparation method.

Under K_0 conditions, the K_0 values obtained from the MT specimens are generally lower than those from the WS specimens. Furthermore, within the range of void ratios tested, the variation in the K_0 value of the MT specimens is much larger compared to that of the WS specimens, as is shown in Fig. 4.

Under undrained conditions, the lower deviatoric stress and the more positive excess pore water pressure are measured at the peak state of the MT specimen. In other words, more contractive behavior is observed for the MT sand. This is consistent with the observations made previously under axisymmetric conditions, which showed a higher liquefaction susceptibility of moist-tamped specimens than those of water- or air-pluviated specimens (e.g., DeGregorio 1990; Hird and Hassona 1990; Vaid et al. 1999; Chu et al. 2003a; Eliadorani and Vaid 2003). On the other hand, the results presented in this study have shown that the asymptotic state (Gudehus et al. 1977; Chu and Lo 1994) and, consequently, the slope of constant stress ratio line (Chu et al. 2003b) approached by the effective stress paths of medium loose (or denser) sand are not affected by the specimen preparation method. Similar observations were made under axisymmetric conditions by Verdugo and Ishihara (1996) and Zlatovic and Ishihara (1997), who showed that the steady state of loose sand in triaxial compression was not affected by the specimen preparation method.

Under drained conditions, the medium loose plane-strain specimen prepared by the MT method behaved more contractively than that prepared by the WS method [Fig. 6(*c*)]. As a result, the dilatancy behaviors of the MT and WS specimens were different [Fig. 6(*d*)]. Moreover, similar to an undrained condition, the lower peak deviatoric stress was obtained for the MT specimen under a drained condition [Fig. 6(*a*)]. Despite the differences in stress-strain and dilatancy behaviors of the MT and the WS specimens, the same failure line with the gradient $\eta_f=1.41$ (corresponding to $\phi_f'=44.5°$) was reached in the two tests, as shown in Fig. 6(*b*). From a practical point of view, this is a very important observation because it shows that the effective stress ratio at failure or the effective friction angle of a granular soil should not be affected by the specimen preparation method. Therefore, a relationship between the effective friction angle and the void ratio established for the MT sand should also be applicable for the WS sand, and vice versa. However, it should be emphasized that the stress-dilatancy relationships established for specimens prepared by different reconstitution methods will be different [Fig. 6(*d*)].

The data presented in this paper also illustrate that the plane-strain compression behavior of medium loose sand in strain path testing can be affected by the specimen preparation method. However, the differences in stress-strain behavior of sand will also depend on the strain increment ratio ($d\varepsilon_v/d\varepsilon_1$) imposed on specimens. For example, different stress-strain curves of the MT and WS specimens were obtained from $d\varepsilon_v/d\varepsilon_1=-0.2$ strain path tests. A contractive behavior was obtained for the MT specimen whereas dilative behavior was obtained for the WS specimen, as shown in Fig. 8(*a*). On the other hand, a very similar stress-strain behavior of the MT and WS sand was observed in $d\varepsilon_v/d\varepsilon_1=-0.6$ strain path tests (i.e., strain softening) or in $d\varepsilon_v/d\varepsilon_1=0$ (undrained) strain path tests (i.e., strain hardening followed by banding softening), as shown in Figs. 7(*a*) and 9(*a*).

The difference in the effects of the specimen preparation method on the stress-strain behavior of Changi sand in strain path testing can be explained by using the conditions for the occurrence of strain softening established by Chu et al. (1992) under axisymmetric conditions. As shown by Chu et al. (1996) and Wanatowski (2005), the conditions for the occurrence of strain softening are also applicable to true triaxial and plane-strain tests. Chu et al. (1992) have reported that whether a soil element undergoes strain softening or hardening depends on the relative magnitude of the strain increment ratio of soil at failure obtained from a drained test, $(d\varepsilon_v/d\varepsilon_1)_f$, and the strain increment ratio, $(d\varepsilon_v/d\varepsilon_1)_i$, imposed during the test. When the strain increment ratio imposed on the specimen, $(d\varepsilon_v/d\varepsilon_1)_i$, is larger (i.e., more positive) than a strain increment ratio at failure as measured in a drained test, $(d\varepsilon_v/d\varepsilon_1)_f$, strain hardening behavior will prevail. On the other hand, when the $(d\varepsilon_v/d\varepsilon_1)_i$ is smaller (i.e., more negative) than the strain increment ratio at failure as measured in a drained test, $(d\varepsilon_v/d\varepsilon_1)_f$, strain softening will occur. However, as shown in this study, different $(d\varepsilon_v/d\varepsilon_1)_f$ ratios will be obtained from drained tests conducted on two similar specimens prepared by different reconstituting methods [Fig. 6(*d*)]. Therefore, in order to determine whether a specimen prepared by a given reconstituting method will undergo strain softening in a $(d\varepsilon_v/d\varepsilon_1)_i=\text{const}$ test, the relative magnitude of the $(d\varepsilon_v/d\varepsilon_1)_i$ and $(d\varepsilon_v/d\varepsilon_1)_f$ obtained from a drained test conducted on a similar specimen reconstituted by the same method must be determined.

For instance, in the $d\varepsilon_v/d\varepsilon_i=-0.2$ test on the WS sand $(d\varepsilon_v/d\varepsilon_1)_i=-0.20$ and $(d\varepsilon_v/d\varepsilon_1)_f=-0.25$ [see Fig. 6(*d*)]. Consequently, $[(d\varepsilon_v/d\varepsilon_1)_i-(d\varepsilon_v/d\varepsilon_1)_f]=0.05$ and strain hardening behavior prevails in the pre-failure region of the WS test. However, in the $d\varepsilon_v/d\varepsilon_1=-0.2$ test on the MT sand, $(d\varepsilon_v/d\varepsilon_1)_f=-0.12$ [see Fig. 6(*d*)] and $[(d\varepsilon_v/d\varepsilon_1)_i-(d\varepsilon_v/d\varepsilon_1)_f]=-0.08$. Therefore, pre-failure strain softening is observed for the MT sand. Using the same framework, a stress-strain behavior observed in the $d\varepsilon_v/d\varepsilon_i=0$ (undrained) or the $d\varepsilon_v/d\varepsilon_i=-0.6$ test can be analyzed. A relative magnitude of the $(d\varepsilon_v/d\varepsilon_1)_f$ and the $(d\varepsilon_v/d\varepsilon_1)_i$ measured in the $d\varepsilon_v/d\varepsilon_i=0$ (undrained) tests is positive for both MT and WS specimens. As a result, the strain hardening behavior was observed in both tests (Fig. 7). In contrast, a negative relative magnitude of the $(d\varepsilon_v/d\varepsilon_1)_f$ and the $(d\varepsilon_v/d\varepsilon_1)_i$ was measured in both $d\varepsilon_v/d\varepsilon_1=-0.6$ tests conducted on the MT and the WS specimens. Therefore, strain softening occurred in these two tests (Fig. 9).

In general, the differences in the stress-strain behaviors of moist-tamped and water-pluviated sand are due to different soil fabrics and structures produced by each method (Vaid et al. 1999; Frost and Jang 2000; Frost and Park 2003; Yamamuro and Wood 2004). Such differences in the fabric and structures of granular soils are related to different deposition and densification processes involved in different preparation methods. In this study, the MT method involved placing moist sand in several layers, whereas the WS method involved raining dry sand through water. Furthermore, the vertical stresses applied during preparation of the MT and WS specimens were different. In the case of loose and medium loose specimens, tamping had to be used for the MT specimens, but not for the WS specimens. In the case of medium dense specimens, tamping was used for both MT and WS specimens. However, for the MT specimens, the tamping effort was much greater than for the WS specimens. These differences, in turn, affect the global uniformity and the grain contact structure of specimens. As shown by

Vaid et al. (1999), Frost and Jang (2000), and Frost and Park (2003), the moist-tamped specimens are always more nonuniform in void ratio distribution than the water-pluviated specimens. Work by Yamamuro and Wood (2004) also suggests that moist-tamped specimens might retain more unstable grain contacts than water-pluviated specimens. In other words, an unstable and highly compressible particle microstructure is produced by the MT. Therefore, although a direct examination of the sand fabric has not been carried out, it can be concluded that the differences in the stress-strain behaviors of the MT and WS specimens observed in this study may be related to the differences in soil fabrics and structures resulting from different specimen preparation methods.

Conclusions

The results of K_0 consolidated drained, undrained, and strain path plane-strain compression tests conducted on medium loose specimens prepared by two different preparation methods are presented and compared in this study. Based on the experimental results presented, the following conclusions can be drawn:

1) The plane-strain compression behavior of medium loose sand under K_0, drained, and strain-path controlled (including undrained) conditions is affected by the specimen preparation method.
2) The data show that the K_0 values obtained from the tests on MT specimens form a relationship with the initial void ratio. However, the K_0 values obtained from the tests on WS specimens demonstrate little dependence on the initial void ratio. The K_0 values obtained from the MT specimens are generally lower than those obtained from the WS specimens.
3) Under drained conditions, specimens prepared by the MT method behave more contractively that those prepared by the WS method. As a result, the dilatancy behaviors of the MT and WS specimens are different. Despite the differences in stress-strain and dilatancy behaviors of the MT and the WS specimens, the same failure line is reached in the two tests. This indicates that the effective stress ratio at failure or the effective friction angle of a granular soil should not be affected by the specimen preparation method.
4) Under undrained conditions, the lower peak deviatoric stress and the higher excess pore water pressure are obtained for the MT sand. However, the results presented in this study show that the asymptotic state (Gudehus et al. 1977; Chu and Lo 1994) approached by the effective stress paths of medium loose (or denser) sand is not affected by the specimen preparation method.
5) Although plane-strain compression behavior of medium loose sand in strain path testing can be affected by the specimen preparation method, the differences in the stress-strain behavior of sand will also depend on the strain increment ratio $(d\varepsilon_v/\varepsilon_1)$ imposed on specimens. Therefore, in order to determine whether contractive or dilative behavior will be observed in a strain path test, the strain increment ratio, $(d\varepsilon_v/d\varepsilon_1)_i$, imposed on a specimen and the strain increment ratio at failure obtained from a drained test, $(d\varepsilon_v/\varepsilon_1)_f$, have to be compared, as suggested by Chu et al. (1992).
6) The difference in the stress-strain behaviors of the WS and MT specimens can be related to the differences in the soil fabrics and structures resulting from different specimen preparation methods.

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