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 ($\delta e_2/\delta e_1$) 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 Mullilis et al. (1977). They reported that different specimen preparation procedures significantly affected the liquefaction characteristics of sand in undrained stress-controlled cyclic triaxial tests. Mullilis 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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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, \( \frac{d\varepsilon}{d\sigma} \), 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 \((\sigma_t)\) 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 void ratio of curvature is not the highest, as often assumed, but it is in between that for a dilative ($d_{e_v}/d_{e_1}>0$) strain path and a compressive ($d_{e_v}/d_{e_1}<0$) 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 a similar way to loose sand when it is subjected to an adequate dilative strain path, as observed by Chu et al. (1992), Vaid and Elia-dorani (1998), and Lancelot et al. (2004). As such, only undrained ($d_{e_v}/d_{e_1}=0$) and dilative ($d_{e_v}/d_{e_1}=-0.2$ and $-0.6$) strain path tests were carried out in the present study.

It needs to be pointed out that the pore water pressure developed in an undrained shear test ($d_{e_v}/d_{e_1}=0$) is not the highest, as often assumed, but it is in between that for a dilative ($d_{e_v}/d_{e_1}>0$) strain path and a compressive ($d_{e_v}/d_{e_1}>0$) 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 a similar way to loose sand when it is subjected to an adequate dilative strain path, as observed by Chu et al. (1992), Vaid and Elia-dorani (1998), and Lancelot et al. (2004). As such, only undrained ($d_{e_v}/d_{e_1}=0$) and dilative ($d_{e_v}/d_{e_1}=-0.2$ 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

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

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

where $\sigma_1$, $\sigma_2$, and $\sigma_3$ 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 $K_0$D 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 $p'_0 = 200$ kPa. Void ratios at the end of $K_0$ consolidation were $e'_v = 0.686$ and $e'_c = 0.681$, respectively. Both specimens were brought to the failure state by a $d_{e_1} = 0$ 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 ($\sigma_2$) in all the plane-strain tests was calculated as an average value obtained from...
four individual pressure transducers installed in the two vertical platen.

Therefore, it was essential to ensure that $\sigma_2$ values measured by the four individual transducers were consistent and reliable. Typical $\sigma_2$ versus $\varepsilon_1$ 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 $\sigma_2$ will diverge. Furthermore, the point where the $\sigma_2$-$\varepsilon_1$ 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.

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

<table>
<thead>
<tr>
<th>Test</th>
<th>Preparation Method</th>
<th>Type of Test</th>
<th>Initial State $^a$</th>
<th>$K_0$ consolidated State</th>
<th>Peak State</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{K0D13}$</td>
<td>WS</td>
<td>drained (d$\sigma_2$=0)</td>
<td>$\varepsilon_{0p}$</td>
<td>$\varepsilon_{p}$</td>
<td>$\sigma_{0p}$</td>
</tr>
<tr>
<td>$C_{K0D15}$</td>
<td>WS</td>
<td></td>
<td>0.689  59.3</td>
<td>0.686  60.1</td>
<td>200.5  223.7  0.59  0.36</td>
</tr>
<tr>
<td>$C_{K0D08}$</td>
<td>MT</td>
<td></td>
<td>0.983  17.5</td>
<td>0.914  0.5</td>
<td>202.0  166.0  3.41  0.47</td>
</tr>
<tr>
<td>$C_{K0Dws}$</td>
<td>WS</td>
<td></td>
<td>0.738  46.5</td>
<td>0.725  49.9</td>
<td>200.2  186.4  0.73  0.37</td>
</tr>
<tr>
<td>$C_{K0Dmt}$</td>
<td>MT</td>
<td></td>
<td>0.741  45.7</td>
<td>0.722  50.7</td>
<td>201.6  219.5  1.08  0.25</td>
</tr>
<tr>
<td>$C_{K0Uws}$</td>
<td>WS</td>
<td>Undrained strain path</td>
<td>0.761  40.5</td>
<td>0.734  47.5</td>
<td>199.2  205.0  0.85  0.34</td>
</tr>
<tr>
<td>$C_{K0Umt}$</td>
<td>MT</td>
<td>(d$\varepsilon_2$/d$\varepsilon_1$=0)</td>
<td>0.772  37.6</td>
<td>0.739  46.2</td>
<td>196.8  229.6  1.14  0.28</td>
</tr>
<tr>
<td>$C_{K0SP−0.2ws}$</td>
<td>WS</td>
<td>Dilative strain path</td>
<td>0.746  44.4</td>
<td>0.730  48.6</td>
<td>202.7  191.9  0.91  0.39</td>
</tr>
<tr>
<td>$C_{K0SP−0.2mt}$</td>
<td>MT</td>
<td>(d$\varepsilon_2$/d$\varepsilon_1$=−0.2)</td>
<td>0.755  42.0</td>
<td>0.735  47.3</td>
<td>200.0  210.4  1.17  0.26</td>
</tr>
<tr>
<td>$C_{K0SP−0.6ws}$</td>
<td>WS</td>
<td>Dilative strain path</td>
<td>0.764  39.7</td>
<td>0.745  44.6</td>
<td>200.7  200.0  1.10  0.38</td>
</tr>
<tr>
<td>$C_{K0SP−0.6mt}$</td>
<td>MT</td>
<td>(d$\varepsilon_2$/d$\varepsilon_1$=−0.6)</td>
<td>0.759  41.0</td>
<td>0.738  46.5</td>
<td>203.6  216.3  1.19  0.25</td>
</tr>
<tr>
<td>$K_0MT1$</td>
<td>MT</td>
<td>$K_0$ consolidation</td>
<td>0.765  39.4</td>
<td>0.750  43.3</td>
<td>202.2  232.7  0.86  0.25</td>
</tr>
<tr>
<td>$K_0MT7$</td>
<td>MT</td>
<td>(d$\varepsilon_2$/d$\varepsilon_1$=1.0)</td>
<td>0.978  −16.2</td>
<td>0.915  0.3</td>
<td>198.0  157.2  3.29  0.48</td>
</tr>
<tr>
<td>$K_0WS4$</td>
<td>WS</td>
<td></td>
<td>0.768  38.6</td>
<td>0.756  41.8</td>
<td>202.9  203.5  0.93  0.39</td>
</tr>
</tbody>
</table>

$^a\sigma_{0p}$=420 kPa, $\sigma_{0p}$=400 kPa, $\sigma_{0p}$=20 kPa, $\sigma_{0p}$=0 kPa

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

*K₀* Consolidation

The *K₀* 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/σ₁₂ and *σ*₃/σ₁₂ versus axial strain curves obtained from a *K₀* 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, *σ*₂/σ₁₂ = *σ*₃/σ₁₂ is obtained under the *d*₂/σ₁₂ = *d*₃/σ₁₂ = 0 condition. Similar observations were made from all the other *K₀* consolidation tests conducted on Changi sand in the plane-strain apparatus. This suggests that there is no strong anisotropy in the *σ*₂/σ₁₂ and *σ*₃/σ₁₂ 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 *K₀* 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 plot determined for Changi sand.

\[
K₀ = 1 - \sin \phi'
\]  

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

It can also be seen from Fig. 4 that the *K₀* values obtained from tests on the MT specimens are different from those on the WS specimens at the same void ratio. The *K₀* 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 *σ*₋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 *K₀* value and the lower axial strain were measured during *K₀* consolidation of the MT sand. It should also be noted that a *K₀* test on sand has to be started from an initial isotropic stress of 20 kPa. This is because the *K₀* 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 *K₀* state. However, this transition only affects the *K₀* value at the initial period. The *K₀* 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 *K₀* path when the consolidation stress is beyond four times the initial stress.

The effective stress paths obtained from the two *K₀* tests are plotted in Fig. 5(b). It can be seen that the effective *K₀* paths obtained from the WS and MT specimens are different. This shows that the specimen preparation method affects not only the *K₀* value

FIG. 2—The *σ*₂ versus *ε*₁ curves obtained for a very loose specimen prepared by the moist tamping method.

FIG. 3—The lateral stress response obtained from a *K₀* consolidation test.

FIG. 4—*K₀* versus *ε*₀ plot determined for Changi sand.

FIG. 5(a)—Effective stress paths from *K₀* tests on WS and MT specimens with similar void ratios.
of sand but also the effective stress ratio \( \eta_v \) determined at the end of \( K_0 \) consolidation. As shown in Fig. 5(b), the higher effective stress ratio \( \eta_v \) 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 \( \eta_v \) 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 \( \sigma'_f = 526 \) kPa 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 gradually reduced 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 \( q_p = 184 \) kPa for the WS sand and \( q_p = 219 \) kPa for the MT sand). This is because the effective stress ratio at the end of \( K_0 \) consolidation, \( \eta_v \), obtained from the MT specimen is higher than that from the WS specimen, as explained earlier. Nevertheless, the same failure line with the gradient \( \eta_v = 1.41 \) 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 \( e_v - e_1 \) curves of the two tests are compared in Fig. 6(c). The two \( e_v - e_1 \) 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 \( (de_v/de_1) \) versus axial strain \( (e_1) \) curves are plotted. In both tests, a contractive behavior \( (i.e., de_v/de_1 > 0) \) was observed until the characteristic state \( (Luong 1980) \) was reached. After that a dilative behavior \( (i.e., de_v/de_1 < 0) \) 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 \( e_v - e_1 \) 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 \( (de_v/de_1)_f = -0.12 \) and \( (de_v/de_1)_f = -0.25 \) 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 \( e_1 \) 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 \( (de_v/de_1 = 0) \)**

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 \( de_v/de_1 \) 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
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 $\eta_{\text{asy}}=1.37$. 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 $\Delta u-e_1$ 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 $e_1$ 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 ($d\varepsilon_1/d\varepsilon_1 < 0$)**

The results of two dilative strain path tests with $d\varepsilon_1/d\varepsilon_1 = -0.2$ 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 $d\varepsilon_1/d\varepsilon_1 = -0.2$ 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-
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
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 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 (\( \eta_{asy} = 1.37 \)), as shown in Fig. 7(b).

The \( \Delta \mu-e_1 \) 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(h)], 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 \( d\varepsilon_p/d\varepsilon_1 = -0.2 \) 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 \( d\varepsilon_p/d\varepsilon_1 = -0.6 \) 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 \( p'_v = 200 \) kPa and then sheared with \( d\varepsilon_p/d\varepsilon_1 = -0.6 \) 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 \( q-e_1 \) and \( q-p'_v \) 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, \( \eta_0 \), 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 \( \Delta \mu-e_1 \) and \( b-e_1 \) 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

FIG. 9—Comparison of dilative \((e_1/e_1 = -0.6)\) strain path tests conducted on WS and MT specimens.
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; Elia- dorani 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. 1997; 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 \( \phi_f = 44.5^\circ \)) 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 \( (d_{e_3}/d_{e_1}) \) imposed on specimens. For example, different stress-strain curves of the MT and WS specimens were obtained from \( d_{e_3}/d_{e_1} = -0.2 \) 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 \( d_{e_3}/d_{e_1} = -0.6 \) strain path tests (i.e., strain softening) or in \( d_{e_3}/d_{e_1} = 0 \) (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, \( (d_{e_3}/d_{e_1})_f \), and the strain increment ratio, \( (d_{e_3}/d_{e_1})_i \), imposed during the test. When the strain increment ratio imposed on the specimen, \( (d_{e_3}/d_{e_1})_i \), is larger (i.e., more positive) than a strain increment ratio at failure as measured in a drained test, \( (d_{e_3}/d_{e_1})_i \), strain hardening behavior will prevail. On the other hand, when the \( (d_{e_3}/d_{e_1})_i \) is smaller (i.e., more negative) than the strain increment ratio at failure as measured in a drained test, \( (d_{e_3}/d_{e_1})_i \), strain softening will occur. However, as shown in this study, different \( (d_{e_3}/d_{e_1})_i \) 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 \( (d_{e_3}/d_{e_1})_i = \text{const} \) test, the relative magnitude of the \( (d_{e_3}/d_{e_1})_i \) and \( (d_{e_3}/d_{e_1})_i \) obtained from a drained test conducted on a similar specimen reconstituted by the same method must be determined.

For instance, in the \( d_{e_3}/d_{e_1} = -0.2 \) test on the WS sand \( (d_{e_3}/d_{e_1})_i = -0.20 \) and \( (d_{e_3}/d_{e_1})_i = -0.25 \) [see Fig. 6(d)]. Consequently, \([d_{e_3}/d_{e_1}]_i = (d_{e_3}/d_{e_1})_i = 0.05 \) and strain hardening behavior prevails in the pre-failure region of the WS test. However, in the \( d_{e_3}/d_{e_1} = -0.2 \) test on the MT sand, \( (d_{e_3}/d_{e_1})_i = -0.12 \) [see Fig. 6(d)] and \([d_{e_3}/d_{e_1}]_i = (d_{e_3}/d_{e_1})_i = -0.08 \). Therefore, pre-failure strain softening is observed for the MT sand. Using the same framework, a stress-strain behavior observed in the \( d_{e_3}/d_{e_1} = 0 \) (undrained) or the \( d_{e_3}/d_{e_1} = 0.6 \) test can be analyzed. A relative magnitude of the \( (d_{e_3}/d_{e_1})_i \) and the \( (d_{e_3}/d_{e_1})_i \), measured in the \( d_{e_3}/d_{e_1} = 0 \) (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 \( (d_{e_3}/d_{e_1})_i \) and the \( (d_{e_3}/d_{e_1})_i \) was measured in both \( d_{e_3}/d_{e_1} = -0.6 \) tests conducted on the MT and the WS specimens. Therefore, strain softening occurred in these two tests (Fig. 9).
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 ($d
\varepsilon_2 / \varepsilon_1$) 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, $(d
\varepsilon_2 / d
\varepsilon_1)_p$, imposed on a specimen and the strain increment ratio at failure obtained from a drained test, $(d
\varepsilon_2 / \varepsilon_1)_f$, 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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References


