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Radial Strain Behaviors and Stress State Interpretation of Soil Under Direct Simple Shear

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ABSTRACT

Radial strain behaviors and stress state interpretation methods of silica sand consolidated at different $K_0$ values under modified direct simple shear were investigated in this research. Consolidated drained triaxial compression test, direct shear box test, and wire reinforced direct simple shear tests were conducted and the test results were compared and linked with the modified direct simple shear tests. Based on the observed testing data, simple shear specimens with constant lateral stress exhibited relative small radial strain during shear, especially under small shear strains. After the peak shear resistance has mobilized, radial strain increased rigorously until the specimen failed. Confining pressure and $K_0$ values were found to have great deal of influence on the simple shear radial change. Two different methods were adopted to interpret the stress conditions within the simple shear specimen and the “pure shear” method was found more appropriate in interpreting the stress state in the modified direct simple shear test.

Keywords

direct simple shear, radial strain behavior, pure shear, $K_0$

Nomenclature

\[ k = \text{a constant which is } k = 1 - K_0 \]
\[ K_0 = \text{coefficient of lateral earth pressure at rest, which is } K_0 = \frac{\sigma_h}{\sigma_v} \]
\[ \sigma_h = \text{applied horizontal stress} \]
\[ \sigma_v = \text{applied vertical stress} \]
\[ \sigma_1 = \text{major principal stress} \]
\[ \sigma_3 = \text{minor principal stress} \]
\[ \tau_h = \text{measure horizontal shear stress} \]
\[ \tau_{\text{max}} = \text{maximum shear stress} \]
\[ \varphi = \text{inclination angle of the major principal stress} \] axis to the vertical direction
\[ \phi_u = \text{the inner particle friction angle} \]

**Introduction**

The direct simple shear apparatus is fairly common testing equipment in soil labs. It is usually adopted by experimenters in measuring the soil strength, stress–strain behaviors, and liquefaction resistance [1–6]. Kjellman [7] developed the first simple shear device at the Swedish Geotechnical Institute. The soil specimen in his apparatus was 20 mm in height and 60 mm in diameter. The specimen was confined both by a rubber hose and, outside the hose, by a series of aluminum rings. Later, Bjerrum and Landva [8] from the Norwegian Geotechnical Institute (NGI) modified the Swedish simple shear apparatus by replacing the rings with a spiral of wound wires reinforced outside the membrane. In recent decades, servo controlled load and pressures, computer aided operation program and digitized reading systems, and cyclic loadings have been introduced to the direct simple shear apparatus that have made the device capable of modeling many field conditions; for example, the loading conditions that a seismic shear wave propagates during earthquake.

Theoretically, the NGI simple shear device claims a condition of plane–strain because the specimen is enclosed by a constrained boundary—the wires reinforced membrane/rings. However, under the so-called plane strain condition, neither uniform stress nor uniform strain can be achieved [9,10]. Another issue associated with the NGI simple shear configuration is that it cannot apply pure shear to a specimen due to the missing of the “complementary shear stresses” on the vertical sides. The specimen would have both rocking and coupling issues when under shearing. In the simple shear phase, only normal and shear stresses are measured. Lateral stress is assumed at \( K_0 \) condition. Conventionally, the stress state inside the specimen is interpreted based on a two dimensional model–plane strain. The soil specimen is a three dimensional element; however, any approximation of \( K_0 \) will yield an erroneous result because \( K_0 \) is not constant during shearing. Both Dyvik and Zimmie [11] and Dobry et al. [12] measured the lateral stress and its variation during simple shear. They concluded that the total stress ratio, \( \sigma_3 / \sigma_0 \), approached unity in monotonic loading and may be larger than unity during cyclic loading. The lateral stress measurements, however, were quite complex.

Despite many critical views put forward by the previous researchers, advancements have been made to the direct simple shear device by many practitioners. The introduction of servo controlled cyclic loading, cell pressure, and bidirectional shear loading made the direct simple shear apparatus ever more popular in modeling liquefaction behaviors [13–18]. Although non-uniform stress distributions were existed within the simple shear specimen, the significance of the simple shear apparatus in geotechnical engineering was convinced by the previous researchers [13,19–25]. In this research, the wires of the NGI simple shear device were replaced by cell pressure and the circumferential deformation was monitored by a circumferential deformation device; thus a clear and complete stress–strain boundary had been defined. The specimen was enclosed by a plain latex membrane which without the use of wire reinforcement and the lateral stress was maintained constant. The advantage of this approach was that a truly \( K_0 \) condition could be maintained and the specimen crosssection area change could be corrected. Because there was no restriction laterally and the radial direction was subjected to a free space, the new test configuration was treated as a “triaxial” test that had a clearly defined stress–strain boundary at any phase during shear. The shear stresses on the horizontal planes were directly measured and the stress-strains were calculated based on the monitored radial strain values.

In summary, the objectives of this research are to: (1) examine the radial behavior changes of the soil under simple shear; (2) systematically analyze two stress interpretation methods that were commonly adopted for reducing the simple shear test data; (3) compare and validate the test results by conducting consolidated drained triaxial compression test, direct shear box test, and wires reinforced direct simple shear test; (4) investigate the \( K_0 \) effects on the stress–strain behaviors.

**Methods of Stress State Interpretation**

On the basis of both the stresses and the strains distributing uniformly, two methods exist for interpreting the stress state within a direct simple shear test specimen. When applying the first method, the soil specimen is subjected to “pure shear,” as shown in both Figs. 1 and 2. The horizontal stress, \( \sigma_h = K_0 \sigma_v \), is achieved by the \( K_0 \) consolidation. The second method was proposed by Oda and Konishi [26]. The principal stress axis has a relationship with the vertical direction, \( \tau / \sigma_v = k \tan \varphi \). \( k \) is the shear stress on the horizontal plane, \( \sigma_v \) is the normal stress on the vertical plane, \( k \) is a constant determined only by the inner particle friction angle of \( \phi_u \) of the granular material, and \( \varphi \) is the angle between principal stress direction and the vertical direction. These methods have been widely used by liquefaction researchers in interpreting the stress state in simple shear specimens [27–32].

**METHOD 1**

The stress conditions are assumed in pure shear; thus the principal stresses will change during simple shear. Both \( \Delta \sigma_1 \) and
\( \Delta \sigma_3 \) are of equal magnitude, as shown in the Mohr circles in Fig. 2. Since \( \sigma_3 \) and \( \sigma_2 (\sigma_0 = K_0 \sigma_v) \) are kept constant during shear, the principal stresses and maximum shear stress can be represented as follows:

\[
(1) \quad \sigma_1 = \frac{1}{2} (1 + K_0) \sigma_v + \sqrt{\frac{\sigma_v^2 (1 - K_0)^2}{4} + \tau_h^2}
\]

represents the major principal stress,

\[
(2) \quad \sigma_3 = \frac{1}{2} (1 + K_0) \sigma_v - \sqrt{\frac{\sigma_v^2 (1 - K_0)^2}{4} + \tau_h^2}
\]

represents the minor principal stress,

\[
(3) \quad \tau_{\text{max}} = \sqrt{\frac{\sigma_v^2 (1 - K_0)^2}{4} + \tau_h^2}
\]

represents the maximum shear stress.

The inclination angle of the major principal stress axis to the vertical direction, \( \phi \), is expressed as,

\[
(4) \quad \tan \phi = \frac{\sqrt{\frac{(1 - K_0)^2}{4} + \left( \frac{\tau_h}{\sigma_v} \right)^2} - 1 - K_0}{\left( \frac{\tau_h}{\sigma_v} \right)}
\]

\[\text{FIG. 1} \quad \text{Stress state in pure shear.}\]

\[\text{FIG. 2} \quad \text{Mohr circles in pure shear—method 1.}\]

\[\text{FIG. 3} \quad \text{Mohr circles in method 2.}\]

**METHOD 2**

The core assumption of this method is that

\[
(5) \quad \frac{\tau}{\sigma_v} = k \tan \phi
\]

where \( k = 1 - K_0 = \sin \phi_s \), and \( \phi_s \) is the inner particle friction angle. Oda and Konishi [26] achieved this equation based on the distribution law of contact force. They concluded that this method is valid to predict the mean value of contact force. This method can also be used to obtain the relationships among the granular mass sheared in the simple shear apparatus. Oda and Konishi [26] claimed that the principal axes of both stress and strain increment did not generally coincide with each other, at least up to the peak stress ratio. The maximum shear stress ((\( \tau_{\text{max}} \)), principal stresses ((\( \sigma_1 \) and \( \sigma_2 \))) are represented below:

\[
(6) \quad \tau_{\text{max}} = \frac{(1 - K_0) \sigma_v^2 + \tau_h^2}{2 \sigma_v (1 - K_0)}
\]

represents the maximum shear stress,

\[
(7) \quad \sigma_1 = \frac{(1 - K_0) \sigma_v^2 + \tau_h^2}{\sigma_v (1 - K_0)}
\]

represents the major principal stress, and

\[
(8) \quad \sigma_3 = K_0 \sigma_v
\]

represents the minor principal stress.

Based on this assumption, the horizontal stress which was applied by the rings/wires can be interpreted as

\[
(9) \quad \sigma_h = \frac{1}{k} \left( \frac{\tau_h}{\sigma_v} \right)^2 \sigma_v + (1 - k) \sigma_v
\]

according to Eqs 5–9, the minor principal stress is found to be constant. In addition, the \( K_0 \) value will be completely changed because the value of \( \sigma_v \) is increasing during shear. Therefore, the Mohr circles are expanded eccentrically in \( s-t \) stress space, as shown in Fig. 3.
Material and Methodology

Round-shaped silica sand, produced by US Silicon with a hardness of 7 and a specific gravity of 2.65, was adopted in this research. The grain size distribution curve can be seen in Fig. 4 (ASTM D422-63 [33]). According to the Unified Soil Classification System (USCS), the sand is classified as poorly graded sand (ASTM D2487-11 [34]). The maximum and minimum void ratios in accordance with ASTM D4253-00(2006) [35] and ASTM D4254-00(2006)e1 [36], are 0.765 and 0.544, respectively.

The simple shear specimen was 71 mm (2.8 in.) diameter by approximately 17.8 mm (0.7 in.) height. Tests were carried out in dry condition with a split mold adopted for preparing the specimen. A 20 kPa vacuum is applied to hold the specimen stands upright. Upon completing the specimen preparation phase, the circumferential deformation device is mounted around the specimen and isotropic/K₀ consolidation was then begun during which normal and radial change was monitored. All the test specimens were initially prepared with a void ratio of 0.59 (80 % relative density) so that to maintain a dense condition during the test. When the consolidation phase was achieved, the specimen was sheared at a constant deformation of 0.2 mm/min until 20 % shear strain had been reached. During shear, both the normal and the cell pressures were kept constant. Target density was obtained by compacting each layer and tamping around the bottom base. The simple shear apparatus system (Fig. 5) utilized in this study was of the Norwegian Geotechnical Institute (NGI) type manufactured by Geotechnical Consulting & Testing System (GCTS). The design of the simple shear device was based on an external cell wall triaxial system. The advantage of this system was that the confining pressure could be servo controlled. Modifications were completed by mounting a circumferential deformation device around the specimen (details of the design and specifications could be found in Kang and Kang [37]).

A series of consolidated drained (CD) triaxial compression tests and direct shear tests were carried out in this research to obtain the friction angle of the sand under different stress–strain boundary conditions. The triaxial specimens were prepared with the same initial void ratio (0.59) as the simple shear specimen and saturated using back pressure until a B value of 0.95 or higher. After saturation, the specimens were consolidated by different consolidation pressures at 50, 100, and 200 kPa. A strain rate of 0.2 mm/min was used to shear the specimen until 20 % of shear strain or reach the critical state. This strain rate was adopted for all the tests so that to avoid any strain rate effects. Similarly, the direct shear test specimens were also prepared at a dense condition with an initial void ratio of 0.59 and sheared at a constant strain rate of 0.2 mm/min until 20 % shear strain or critical state has been reached. The details of the direct shear test procedures could be found in Kang et al. [38].

Results and Discussions

From a practical point of view, it is important to know the relation between the shear strength values measured form the direct simple shear test and that of measured from in other types of tests; for example, the triaxial compression test. However, it is hard to compare the results between the direct simple shear test and triaxial compression test because the measured $f_s$ on the horizontal plane is neither the maximum shear stress nor is the shear stress on the failure plane [23,39]. Under the modified test set up, the principal stresses were predetermined which made it possible to compare the strength of soils measured in the direct simple shear test and in the triaxial compression test on an equal basis.

A series of CD tests were run on the silica sand and the internal friction angle was achieved (see Table 1). Another series of
direct shear box tests were run on the sand and the internal friction angle based on the direct shear tests were also tabulated in Table 1. Two different types of direct simple shear tests were carried out. “A” series tests were carried out with stacked Teflon rings reinforced outside the membrane. Baxter et al. [40] concluded that the simple shear results from both the wire reinforced membrane and stacked rings were the same. “B” Series tests were carried out under the modified test configuration. Cell pressures were set at 50, 100, and 200 kPa. Two $K_0$ values, $K_0 = 1$ and 0.5 were adopted. The normalized shear stress ratio versus shear strain from series tests “A” are plotted in Fig. 6. All specimens were sheared to 20% shear strain and all the normalized shear stress ratios became constant. The normalized shear stress ratio versus shear strain from series tests “B” at $K_0 = 1$ and 0.5 are displaced in Fig. 7 and Fig. 8. All the normalized shear stress ratios were smaller than 0.6 as shear strain reached 20%.

### TABLE 1  Friction angle determined by different types of shear tests.

<table>
<thead>
<tr>
<th>Types of Shear Test</th>
<th>Friction Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direct Simple Shear Test (“A”)</td>
<td>46.5°</td>
</tr>
<tr>
<td>Direct Shear Test</td>
<td>47.1°</td>
</tr>
<tr>
<td>Modified Direct Simple Shear Test (“B”)</td>
<td>37.6°</td>
</tr>
<tr>
<td>CD Triaxial Compression Test</td>
<td>37.0°</td>
</tr>
</tbody>
</table>

**RADIAL STRAIN CHANGE**

Both radial change and normalized shear stress versus shear strain curves at different $K_0$ values are plotted in Fig. 7 and Fig. 8. The confining stress levels were set at 50, 100, and 200 kPa, respectively. These specimens exhibited similar trends when the shear resistance was normalized by the vertical stress. All the stress–strain curves developed into a common end, as
shown in the plots (Figs. 7 and 8). Radial change showed a distinct trend which is of great interest. No radial strain was detected within approximately 3% shear strain. For specimens at $K_0 = 1$ (Fig. 7), the radial strain changed to negative as the confining pressure increased. Essentially, the diameter of the specimen was decreased as it was being sheared. For the specimens at $K_0 = 0.5$ (Fig. 8), however, the specimen’s radial strain increased under lower confining pressure and almost constant at higher confining pressure. No radial change was observed throughout the entire test under approximately 200 kPa cell pressure. Another interesting finding is that the radial changes were almost zero within 10% strain for all the specimens. After that, radial strain increased rigorously at a slope near 45° under lower confining pressures. At 200 kPa cell pressure, however, there was almost no radial strain occurred.

Based on the observed radial strain testing data, the following conclusions could be achieved. Confining pressure and $K_0$ values all influence the simple shear radial behavior. When simple shear took place, the diameter of the sample decreased as the confining pressure increased. $K_0$ value had a great deal of influence on the radial strain. When $K_0 = 1$, the largest radial strain was approximately 0.03%. When $K_0 = 0.5$, however, the largest radial strain increased to near 1%. The pre-shear induced by the $K_0$ could cause serious radial strain in simple shear. Nevertheless, the final radial strain for all tests did not go beyond about 1% at a shear strain of approximately 20%. This shows that, even without the wires/rings reinforcement, the specimen could maintain a relative circular cross section area where approximated “0” lateral deformation could be achieved.

**STRESS STATE INTERPRETATION**

Equation 3 illustrates that, as the shear stress acting on the horizontal plane increases, the minor principal stress decreases. Therefore, a critical state, $\sigma_3 = 0$ exists when the shear stress ratio $\tau / \sigma$, equals to $\sqrt{K_0}$. If shear stress ratio $\tau / \sigma$ continues to increase, tension cracks will develop within the specimen and $\sigma_3$ will become negative. Sand particles, however, cannot resist tensile force, and shear stress ratio $\tau / \sigma$ can theoretically never go beyond $\sqrt{K_0}$.

Method 1 is considered not appropriate for interpreting the stress state within a wires/rings reinforced simple shear specimen (series tests “A”). As stated, $\sqrt{K_0} = \sqrt{1 - \sin \phi} = \sqrt{1 - \sin 37°} = 0.63$. This value, however, is smaller than the final normalized shear stress ratios of each test which were 0.89, 0.83, and 0.72, respectively. Under this condition, tensile force would develop inside the specimen. Theoretically, rings/wires cannot cause tensions within a sand specimen. Therefore, method 2 was adopted in analyzing the stresses in series tests “A.” Under this interpretation, the minor principal stress was constant during shear. The Mohr circles would expand as major principal stress increased; once the Mohr circle touched the Mohr Coulomb failure envelope, the Mohr circle stopped expanding. Figure 9 shows the relationship between predicted horizontal stress ratios ($\sigma_h / \sigma_v$) and shear strains at different normal stress levels (obtained by Eq 9). As shear was occurring, the horizontal stress increased, with the final stress ratios of approximately 1.3–1.6 (Fig. 9). The increase of the horizontal stress phenomenon was also discovered and reported by Dyvik and Zimmie [11], Dobry et al. [12], and Ishihara et al. [41]. These researchers claimed that the stress ratio $\sigma_h / \sigma_v$ increased to unity under monotonic loading and might larger than unity under cyclic loading.

Method 1 was employed in studying the stress state in series tests “B.” As previously discussed before, the normalized shear stress ratio should be smaller than $\sqrt{K_0} = \sqrt{0.5} = 0.7$ so
that to prohibit tensile forces from developing within the specimen. Both Fig. 7 and Fig. 8 demonstrate that all the final normalized shear stress ratios were smaller than 0.7. According to the Mohr Coulomb failure criterion, the minor principal stress could not shrink to zero. This is because the Mohr circles will touch the Mohr coulomb failure envelop before minor principal stress decreases to zero and, thus, stops shrinking. As a result, the Mohr circle will stop expanding and both the major and minor principal stresses would stay constant.

Compared with the two different methods, method 1, “pure shear,” was found to work for interpreting the stress state in the modified direct simple shear test setup which had a well-defined stress–strain boundary condition and fixed $K_0$. Method 2, proposed by Oda and Konishi [26], was found suitable for interpreting the stress state of simple shear specimens enclosed by wires reinforced membrane. Using the method 1, pure shear, to interpret stress states in a simple shear specimen with wire reinforced membrane may result in errors.

The principal stresses were calculated and the friction angle of the sand is determined (by method 1 and method 2) and presented in Table 1. Friction angle determined by triaxial compression tests and modified direct simple shear tests (Series tests “B”) were similar, which were 37.0° and 37.6°, respectively. The friction angle determined by direct shear tests and Teflon reinforced direct simple shear tests (Series tests “A”) were similar, which were 46.5° and 47.1°, respectively. However, friction angles determined by triaxial compression tests and direct tests were different, similar for the modified direct simple shear tests and the Teflon reinforced direct simple shear tests. The stress states in the direct shear test are similar to the stress conditions in the Teflon reinforced direct simple shear tests. The stress conditions in triaxial compression tests are similar to the modified direct simple shear tests. It is speculated that the difference between the interpreted friction angle values were resulted from the different stress–strain boundary conditions, namely the plane–strain condition and the plane–stress condition.

Conclusions

A brief literature review of the direct simple shear test was presented in this study. Monotonic simple shear tests were carried out on silica sand. The tests included several stacked Teflon rings reinforced direct simple shear tests and several lateral stress constant (fixed $K_0$ condition) modified direct simple shear tests with radial change monitored. Two previous published methods were used to interpret the stress states within the simple shear specimen. Based on the observed test data, although there were no restrictions in the radial direction, simple shear specimens lacking of restrictions outside had a relatively small radial change under small shear strain. At large shear strain, however, radial strain increased intensely until the specimen failed. Confining pressure and $K_0$ values all influence the simple shear radial behavior. The application of the two stress interpretation methods was found to be largely dependent on the test boundaries. Pure shear method worked well in interpreting the stress state in a specimen with constant lateral confining stress. However, the stress state in a wires/rings reinforced specimen cannot be predicted by pure shear because the $K_0$ value changes during the shear phase. Therefore, using the conventionally pure shear method to determine the stress conditions in the wires/rings reinforced simple shear specimen, however, could yield significant errors.

References

Strength Testing of Soils,” ASTM STP 740, R. N. Yong
and F. C. Townsend, Eds., ASTM International, West Con-
in Short Cylinders Subjected to Axial Deformation and Lat-
114–118.
During Static and Cyclic Direct Simple Shear Testing,” Pro-
cedings of the 3rd International Conference on the
Behavior of Off-Shore Structures, Vol. 2, MIT Press,
Shear Tests of Normally Consolidated Offshore
Orino Clay,” Research Report Prepared for INTEVEP,
Los Teques, Venezuela, 1981.
4, 1979, pp. 190–199.
[14] Silver, M. L., Tatsuoka, F., Phukanphan, A., and Avram-
dis, A. S., “Cyclic Undrained Strength of Sand by Triaxial
and Simple Shear Test,” Proceedings of the Specialty Con-
ference on Earthquake Engineering and Soil Dynamics,
1478–1481.
Behavior of Sand Under Irregular Loading,” Soils Found.,
[16] Dyvik, R., Beere, T., Lacasse, S., and Raadim, B., “Compari-
son of Truly Undrained and Constant Volume
Direct Simple Shear Tests,” Geotechnique, Vol. 37, No. 1,
[17] Boulanger, R. W., Chan, C. K., Seed, R. B., Seed, H. B., and
Sousa, J., “A Low Compliance Bidirectional Cyclic Simple
pp. 36–45.
[18] Boulanger, R. W. and Seed, R. B., “Liquefaction of Sand
Under Bidirectional Monotonic and Cyclic Loading,” J.
[19] Lucks, A. S., Christian, J. T., Brandow, G. E., and Hoeg, K.,
“Stress Conditions in NGI Simple Shear Test,” ASCE J.
155–160.
of NGI Simple Shear Apparatus for Cyclic Soil Testing,” Re-
[21] Kovacs, W. D. and Leo, E., “Cyclic Simple Shear of Large
Scale Sand Samples: Effects of Diameter to Height Ratio,
Proceedings of the International Conference on Recent
Advances in Geotechnical Earthquake Engineering and Soil
Dynamics, The University of Missouri-Rolla, Rolla, MO,
[22] Vucetic, M. and Lacasse, S., “Specimen Size Effect in Sim-
ple Shear Test,” J. Geotech. Eng. Div., Vol. 108, No. 12,
[23] Budhu, M., “Non-Uniformities Imposed by Simple Shear
[24] Amer, M. I., Kovacs, W. D., and Aggour, M., “Cyclic Sim-
Simple Shear Devices,” Soils Found., Vol. 27, No. 2, 1987,
pp. 31–41.
[26] Oda, M. and Konishi, J., “Rotation of Principal Stresses in
Granular Material During Simple,” Soils Found., Vol. 14,
No. 4, 1974, pp. 39–53.
Stress Strain Behavior of ‘Wet’ Clay,” Engineering Plastic-
ity, J. Heyman and F. A. Leckie, Eds., Cambridge Uni-
Measuring Soil Liquefaction Characteristics,” ASCE J.
[29] Oda, M., “On the Relation $\tau /\sigma_s = k\tan \phi$ in the Simple
[31] Riemer, M. F. and Seed, R. B., “Factors Affecting Apparent
Undrained Cyclic Simple Shear,” Geotechnique, Vol. 53,
Analysis of Soils, Annual Book of ASTM Standards, ASTM
[34] ASTM D2487-11: Standard Practice for Classification of
Soils for Engineering Purposes (Unified Soil Classification
System, Annual Book of ASTM Standards, ASTM Interna-
tional, West Conshohocken, PA, 2011.
imum Index Density and Unit Weight of Soils Using a Vi-
bratory Table, Annual Book of ASTM Standards, ASTM
Index Density and Unit Weight of Soils and Calculation of
Relative Density, Annual Book of ASTM Standards, ASTM
Tests on Silica Sand,” Marine Georesources & Geotechnol-
y (to be published).
tions on Crushed Rock–Concrete Interface Behaviors,”
[40] Baxter, C. D. P., Bradshaw, A. S., and Ochoa-Lavergne, M.,
“DDS Test Results Using Wire-Reinforced Membranes
of CG Model Foundation,” Proceedings of the 9th Interna-
tional Conference on Soil Mechanics and Foundation