CYCLIC TRIAXIAL TEST OF DYNAMIC SOIL PROPERTIES FOR WIDE STRAIN RANGE

Takeji Kokusho*

ABSTRACT
Dynamic material properties of soil needed for the nonlinear analysis of earthquake response of the ground have usually been measured in the laboratory using the torsional column shear device. The cyclic triaxial test, despite its easier accessibility and handling for practicing engineers, has seldom been looked upon as a rational way to make a reliable measurement of dynamic properties. In this experimental study a highly sensitive gap sensor and a sealed load transducer are introduced inside the triaxial cell to eliminate the frictions of the conventional displacement sensor and the loading piston. A series of cyclic triaxial tests featuring these improvements have been carried out for saturated Toyoura sand with various void ratios and confining pressures, disclosing the following facts.

(i) Reliable measurements of the shear modulus and its strain dependent variation within a wide strain range of the order of $10^{-8}$ to $10^{-3}$ are possible with the improved cyclic loading test, permitting a direct connection with the in situ shear wave velocity.

(ii) An essentially good agreement of the modulus is quantitatively recognizable between the triaxial test and the torsional shear test, implying that the different stress and confining conditions inherent to the individual testing devices give no significant effect on the evaluation of the modulus.

(iii) This test gives a reasonable hysteretic damping ratio as well as its strain dependency although the damping ratios measured by other researchers with different testing devices widely differ from each other indicating the difficulties for evaluating accurate damping ratio.

Key words: damping, deformation, dynamic, repeated load, sandy soil, test equipment, triaxial compression test

IGC: D 7/D 6/D 0

INTRODUCTION
The first laboratory study on the dynamic response of soil material to earthquake waves was conducted some forty years ago when Iida (1938, 1940) started a resonant type test of soil column under very low confining pressures. In the 1960's the resonant column test was considerably refined by a number of workers as Hardin and Richart (1963), Hall and Richart (1963), Hardin (1965), Hardin and Black (1966, 1968, 1969), and Drnevich and Richart (1970), and enabled a reliable measurements of dynamic properties for various kinds of soils. The series of tests performed by the

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above researchers revealed the significant effect of confining pressures as well as void ratios affecting the dynamic shear modulus of soil. The tests also clarified the importance of consolidation time on the shear modulus for cohesive soils. Influences of other factors on the modulus such as initial shear stress and loading frequency were also examined and concluded to be less important. Soil damping was closely studied and found to be hysteretic rather than viscous.

The above-mentioned researches were restricted to a small strain range of less than $10^{-4}$ since the resonant column testing technique can not be applied to the soil tests to obtain the dynamic properties for larger strain levels because of the inelastic nature of the soil materials. The recent developments in numerical analyses for the nonlinear dynamic responses of grounds or soil structures due to strong earthquake motions have dramatically increased the demand for dynamic soil properties corresponding not only to the smaller strain level but also to the larger strain level. In 1970 Hardin and Drnevich developed a low frequency torsional cyclic shear device for a cylindrical soil specimen, and conducted an experimental study with a special emphasis on the effect of strain level on the dynamic soil properties. That revealed the considerable decrease of the modulus and increase of the damping ratio when the strain increased above $10^{-4}$, as well as the significant effect of the confining stress on the changing rate of the modulus and the damping. It also confirmed the modulus for the shear strain equal to $10^{-3}$ measured with the torsional cyclic loading device coincided well with that measured with the resonant column device. In 1971 Silver and Seed made similar tests for sands employing the simple shear apparatus and made some additional findings about the modulus and the damping, and their dependence on strain level. In 1976 Iwasaki, Tatsuoka and Takagi performed a number of experiments on Toyoura sand using the resonant column and torsional cyclic shear devices which were further improved to get a more reliable modulus and its strain-dependent variation.

Thus the torsional shear device, either the resonant column type or the torsional cyclic shear type, has played a key role for the experimental research on the dynamic soil properties for the past two decades. It has also been pointed out that the earthquake-induced stress conditions of the ground can be more properly reproduced in the solid or hollow column specimens of this device than in the other testing devices.

On the other hand the dynamic triaxial test despite its frequent use in the engineering practice has seldom been looked upon as a rational way to make a reliable measurement of dynamic soil properties for small strain level in particular. It belongs to a common knowledge that the triaxial stress condition has less resemblance than the simple shear condition to the situation of insitu ground excited by horizontal shear waves. Namely, in the triaxial specimen the principal stress planes do not continuously rotate, and more significantly the two unsymmetric loading conditions; compression and extension, take place alternatively during each loading cycle. In spite of these disadvantages, the triaxial apparatus has some superiorities which should not be overlooked; it is simpler in mechanism, easier to handle and more economical, and offers greater availability and easier access for most practicing engineers than the other testing devices. In the author's opinion the main reason why the triaxial test has been less authorized as a reasonable means to obtain a good quality dynamic properties stems from the difficulties of making reliable measurements corresponding to a smaller strain level than $10^{-4}$. Most of the moving parts of the triaxial apparatus suffer from mechanical frictions which make the highly sensitive measurement rather meaningless. In other words the properties of deformation and the internal energy loss for soil become mixed with those caused by the mechanical frictions of the triaxial apparatus itself preventing a dependable measurement for the small strain level. It is therefore
essential to eliminate those frictions related with the measurement of displacement and load in order to improve the test.

In this experimental research a new type of highly sensitive displacement sensor completely exempt from frictions and which can detect a strain as small as $10^{-6}$ has been introduced just above the loading cap inside the pressure chamber of the triaxial apparatus. In addition, a load cell has also been installed inside the triaxial chamber to measure the cyclic axial stress free from the effect of the friction of the loading piston. A series of cyclic loading tests for saturated Toyoura sand have been conducted for the wide strain range of the order of $10^{-4}$ to $10^{-5}$, with the improved triaxial device to compare the results with those of other investigators, and to show efficiency and reliability of this experimental method for dynamic soil properties.

**Table 1. Specifications of the gap sensor and the load transducer**

<table>
<thead>
<tr>
<th></th>
<th>Gap sensor</th>
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</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>$2.86 \times 10^{-2}$ m</td>
</tr>
<tr>
<td>Thickness</td>
<td>$0.32 \times 10^{-2}$ m</td>
</tr>
<tr>
<td>Max. sensitivity</td>
<td>$3 \times 10^{-3}$ m per volt</td>
</tr>
<tr>
<td>Resolution</td>
<td>Infinite on recorder output</td>
</tr>
<tr>
<td>Signal to noise ratio</td>
<td>50 to 1</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Load transducer</th>
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</thead>
<tbody>
<tr>
<td>Diameter</td>
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</tr>
<tr>
<td>Thickness</td>
<td>$3.75 \times 10^{-2}$ m</td>
</tr>
<tr>
<td>Capacity</td>
<td>$0.98$ kN</td>
</tr>
<tr>
<td>Compression</td>
<td>Compression and tension type</td>
</tr>
</tbody>
</table>

**Fig. 1. Section of the improved triaxial device**

**Photo. 1. Upper part of the improved triaxial device**
TEST METHOD

The cyclic triaxial device used in the test belongs to a quite usual type with the specimen size of 5 cm in diameter by 10 cm in height, as shown in Fig.1. In order to improve the device so that it can measure the soil properties corresponding to a small strain level of $10^{-4}$ or less, a highly sensitive “gap sensor” and a load transducer the specifications of which are listed in Table 1 were installed in the pressure chamber to eliminate the mechanical frictions caused by the loading piston and the displacement sensor. The gap sensor is composed of two separate electromagnetic discs of 28.6 mm in diameter highly sensitive to the relative distance. The sensor is literally free from any mechanical friction whose maximum output exceeds 0.2 volt per micron. A pair of gap sensors were attached to the both sides of the loading cap diagonally opposite to each other, as shown in Photo.1, to detect the average axial displacement eliminating the component of its uneven movement.

Saturated Toyoura sand chosen as the test material was compacted into the mold with a handy vibrator and then dried completely. The specific gravity, $G_s$, the mean grain size, $D_{50}$, and the uniformity coefficient, $U_p$, of the Toyoura sand are 2.64, 0.19 mm and 1.3 respectively. After isotropically consolidating the sand with a specific confining stress, the back pressure was raised to 98 to 196 kN/m$^2$ (1 to 2 kgf/cm$^2$) in order to attain the Skempton’s $B$ coefficient of greater than 0.96. The parameters varied in the tests were the void ratio ($e=0.64$ to 0.80), the effective confining stress ($\sigma'=19.6$ to $294$ kN/m$^2$ or $\sigma'=0.2$ to $3.0$ kgf/cm$^2$) and the number of loading cycles ($N=1$ to 10). The cyclic axial load was applied to isotropically consolidated specimens by a pneumatic actuator manually controlled to make the cyclic stress-control test. Although the loading frequency was 0.1 Hz to 0.02 Hz which was much lower than that for usual dynamic tests, it may be justified by the previous researches (Hardin, 1965) finding virtually no significant effect of loading rate on the dynamic properties of sands.

In most tests the specimen was loaded under the undrained condition during each loading cycle permitting drainage at the end of each cycle whenever the pore pressure buildup took place. Some additional tests were also conducted in a completely drained condition to compare with the undrained test. It is readily understood that for the triaxial stress condition the modulus of deformation, $E$, and the shear modulus, $G$, can be derived from

$$E = \Delta \sigma / \Delta \varepsilon$$

$$G = \Delta \tau / \Delta \gamma$$

in terms of the axial stress amplitude, $\Delta \sigma$, the axial strain amplitude, $\Delta \varepsilon$, the shear stress amplitude, $\Delta \tau$, and the shear strain amplitude, $\Delta \gamma$. For the constant radial stress these four magnitudes are interrelated as

$$\Delta \tau = \Delta \sigma / 2$$

$$\Delta \gamma = (1 + \nu) \Delta \varepsilon$$

Under the undrained condition Poisson’s ratio, $\nu$, can be almost 0.5 for a completely saturated soil allowing the derivation of shear strain, $\Delta \gamma$, and the shear modulus, $G$, from the above equations while the drained test only gives $\Delta \varepsilon$ and $E$. It is expected in the undrained case that the change of the mean stress, one third of the axial stress change, is entirely carried by the pore pressure change realizing the shear test under constant effective confining stress as long as the excess pore pressure buildup due to dilatancy is not so significant during each loading cycle. Thus the undrained test is more favorable whenever possible since it yields the shear modulus, the more
fundamental material constant than the modulus of deformation.

The electric signals of the axial deformation and the axial load, after having been passed through a wave filter circuit to cut off high frequency noise, were introduced to an X-Y recorder to draw a stress-strain loop as schematically illustrated in Fig. 2 for each cycle. From the loop the equivalent modulus called the secant modulus was determined by the slope of the straight line connecting the two extremes of the loop. The equivalent damping ratio, \( h \), was calculated in terms of the ratio of the area \( \Delta A \) enclosed by the loop and the triangular area \( A \) shown in Fig. 2 by the following equation.

\[
h = \frac{\Delta A}{\pi A}
\]  

Fig. 2. Schematical stress-strain loop

\[
\Delta A = \text{Area under the loop}
\]
\[
A = \text{Area of triangle}
\]
\[
h = \frac{1}{\pi} \cdot \frac{\Delta A}{A}
\]

Fig. 3. Shear modulus vs. shear strain relationship obtained with the conventional triaxial apparatus

RESULTS AND DISCUSSIONS

Preliminary Test

Before starting a series of tests using the improved device, some preliminary cyclic loading tests were carried out with the conventional triaxial apparatus to better understand the problems involved in this testing method. This apparatus was equipped with an usual LVDT as the axial deformation transducer as well as the load transducer outside the pressure chamber. Fig. 3 shows the variation of the shear modulus, \( G \), along with the shear strain, \( \gamma \), on the semi-logarithmic graph. Loose Gifu sand, very similar to Toyoura sand, \( (G_s=2.67, D_{50}=0.29 \text{ mm}, U_s=1.8) \) was used for this test. The open circles on the graph correspond to the moduli directly determined from the hysteresis loops, while the solid circles correspond to the modified moduli determined by taking the error in the stress measurement caused by the friction of the loading piston into consideration. Taking it for granted from other research works that a rational \( G \sim \log \gamma \) curve should have an asymptote when \( \gamma \) is less than \( 10^{-5} \) as illustrated with the solid curve, even the modified modulus corrected for the friction at the piston indicates to include a significant error for small shear strain range. It is quite persuasive to attribute the error to the mechanical friction of the displacement transducer itself which is by no means negligible for the small strain range. The preliminary tests as compared to the improved tests to be discussed in the next section proves the necessity for the improvement of the measuring system in order to obtain the reliable soil properties.

Improved Cyclic Triaxial Test

A number of cyclic loading tests employing the improved triaxial device were per-
Fig. 4. Measured hysteresis loops for (a) the improved tests and for (b) the conventional tests.

Fig. 5. Shear modulus and damping ratio vs. shear strain relationship for different loading cycles.

Fig. 6. $G$ vs. log$\gamma$ relationships for dense sand with different confining pressures.
formed for saturated Toyoura sand. Fig. 4 shows some representative hysteresis loops obtained with the improved device as compared with the conventional device for various strain levels and various numbers of cycles. The loops in Fig. 4(a) are evidently smooth for all strain levels, while those in Fig. 4(b) possess bilinear characteristics for smaller strain levels. This indicates that the improved device is completely free from any mechanical problems of the testing device, and therefore yields high-quality test results even for the small strain range. The shear modulus and the damping ratio measured in the undrained test with the confining pressure $\sigma_c' = 98 \text{kN/m}^2$ (1.0 kgf/cm$^2$) are shown in Fig. 5 for the number of cycles $N = 2, 5$ and 10. It is obviously seen that the modulus converges to a certain value as the strain gets smaller than $10^{-3}$. It is also noted that the number of cycles, $N$, has little effect on the modulus for $N = 2$ to 10, whereas there exists some sizable difference of the damping ratio between $N = 2$ and $N = 5$.
5 or 10. In the subsequent discussions soil properties have all been determined for \(N=10\).

Fig.6 contains curves of the shear modulus versus shear strain relationships on the semi-logarithmic graph obtained from thirteen undrained tests of the dense sand (\(e=0.64\)) for the confining stress ranging from 19.6 to 294 kN/m² (0.2 to 3.0 kgf/cm²). Each curve shows a continuous increase of the modulus as the shear strain decreases for the order of \(10^{-3}\) to \(10^{-5}\) permitting the extrapolation of the modulus at \(\gamma=1\times10^{-4}\) which is denoted here as \(G_0\). The shear modulus, \(G\), in Fig.6 normalized by \(G_0\) for each confining stress is taken against \(\log\gamma\) in Fig.7 to obtain the strain-dependent curve of the modulus. It is apparent that the average curve corresponding to each confining stress shifts consistently to the right along \(\gamma\)-axis as the stress becomes higher. Fig.8 shows the variations of the damping ratio, \(h\), measured in the same tests taken against shear strain. Average \(h\) versus \(\log\gamma\) curves also move to the right with the increase of the confining stress.

In Fig.9 the effect of void ratio on the modulus is examined for the saturated sand with four different void ratios for the confining pressure equal to 98 kN/m² (1.0 kgf/cm²). The normalized modulus ratio, \(G/G_0\), versus \(\log\gamma\) relationships taken from Fig.9 are shown in Fig.10. The damping ratio, \(h\), versus \(\log\gamma\) relationships obtained by the same group of tests are shown in Fig.11. Compared to the confining stress,
void ratio has less influence on the strain dependency curves of the modulus ratio, $G/G_0$, and the damping ratio, $h$, as well, although both curves tend to slightly shift to the left along $\gamma$-axis as the void ratio becomes larger. Also apparent from the figures is that for looser sands this test can not provide the properties for larger strain level disclosing the shortcomings of the triaxial stress system. Namely, compared to the simple shear stress system, the looser specimen in the triaxial stress system reaches failure in the extension mode in a relatively early stage of the test.

Besides the undrained tests mentioned above a couple of drained tests were carried out to examine the effect of drainage conditions on the measurement of dynamic soil properties. This is of importance in engineering sense since the undrained test of fully saturated soils is not always possible in the practical problems because the soils are sometimes only partially saturated. Fig.12 shows the strain-dependent changes of the modulus ratio, $E/E_0$, for four confining stresses along with the increase of axial strain, $\varepsilon$, on the semi-logarithmic graph, while Fig.13 shows the damping ratio, $h$, versus axial strain, $\varepsilon$, relationships. Both $E/E_0$ vs. log $\varepsilon$ curve and $h$ vs. log $\varepsilon$ curve are consistently translated to the right along the $\varepsilon$-axis as the confining stress increases. In Fig.14 the strain-dependency curves of the modulus and the damping are compared.
for drained and undrained conditions. For the undrained tests, in addition to the test results illustrated in Figs.7 and 8 which are again referred to with the solid curves in Fig.14, the results of reverse cyclic undrained tests are shown with the dotted curves. In these tests the cyclic load was applied in such a sequence as first-extension–then-compression in contrast to the usual tests of compression–extension sequence. Comparison of the curves of the modulus and the damping for the two kinds of loading reveals that the difference of the loading sequence has little effect on the strain–dependency curves at least for N=10. In order to compare $E/E_0$ vs. log $\varepsilon$ curves of drained test directly with $G/G_0$ vs. log $\gamma$ curves of undrained tests, it is necessary to estimate the shear strain, $\gamma$, from the axial strain, $\varepsilon$. Here, shear strain was calculated from Eq. (4) assuming $\nu=0.5$ for drained tests, and $E/E_0$ vs. log $\gamma$ curve was derived as shown with the chain-dotted curves in Fig.14. If $\nu$ is assumed to be 0.3 the curves will be located a bit left along the logarithmic axis of $\gamma$ resulting in no significant difference. In any case the two curves of the modulus ratio and the damping for drained and undrained tests are in a relatively good coincidence. This implies that from the engineering point of view drained tests, easier to make than undrained tests, can yield almost analogous strain–dependent variations of the modulus and the damping to those of undrained tests.

Comparison with Other Test Results

In the preceding section the dynamic modulus and the equivalent damping ratio for sand obtained with the improved triaxial device have been closely examined to demonstrate the efficiency of this new testing method. It is of great interest now to compare the test results with those measured with different devices like the torsional shear device or the simple shear device.
The shear moduli, $G_o$, corresponding to $\gamma=1\times10^{-4}$ obtained by a number of undrained cyclic triaxial tests are plotted in Fig.15 against void ratios, $e$. In the figure, these results are then compared with the following equation proposed by Richart, Hall and Woods (1970) for sands with round grains

$$G = 700 \frac{(2.17-e)^2}{1+e} (\sigma'_{e})^{1/2} \quad (6)$$

where the unit of the stress and the modulus is kgf/cm² ($1\text{ kgf/cm}^2 = 98\text{ kN/m}^2$). The figure clearly indicates the triaxial test results will coincide well with the equation for all the confining stresses if the multiplier involved in the equation is increased by 20% as illustrated with the dotted curves giving the new equation

$$G = 840 \frac{(2.17-e)^2}{1+e} (\sigma'_{e})^{1/2} \quad (7)$$

This remarkable agreement may imply that the modulus for small strains corresponding to elastic wave velocity can be readily measured with the low frequency cyclic triaxial test.

![Fig. 16. Modified $G_o$ vs. $\sigma'_e$ relationships on the logarithmic graph for different strain levels](image)

![Fig. 17. Comparison of modified $G_o$ vs. $\sigma'_e$ relationships with those of other researchers](image)

In Fig.16 the shear moduli, $G$, modified with the function of $e$ introduced in Eq. (6) to eliminate the effect of void ratio are taken against the effective confining pressures, $\sigma'_e$, on the logarithmic graph for different strain levels of $10^{-6}$ to $10^{-3}$. The slope of the straight line in the graph approximating $\log G$ vs. $\log \sigma'_e$ relationship for each strain level gets steeper as the strain becomes larger. The group of the lines is then compared in Fig.17 with that obtained by Iwasaki, Tatsuoka and Takagi (1976) for the same saturated Toyoura sand employing the resonant column and cyclic torsional shear devices. It is of great significance to note that the two tests employing quite different testing apparatus gave exactly the same modulus for $\gamma=3\times10^{-4}$ and $\gamma=10^{-3}$. For the strain smaller than $\gamma=10^{-4}$ some appreciable difference of the modulus exists between the two tests which might be explained by the fact that instead of the cyclic
test the resonant column test plays an important role in the research of Iwasaki et al. to decide the modulus and its strain dependency for the strain smaller than $\gamma = 10^{-4}$. Another slight discrepancy between the two tests is visible for the large strain of $\gamma = 3 \times 10^{-3}$ and probably stems from the fact that the disadvantage of triaxial stress condition previously mentioned becomes evident for the larger strain range. In any event it would be of particular interest to understand that the different stress and confining conditions inherent to the individual testing devices have only an insignificant effect on the measurement of the modulus.

The group of the straight lines drawn in Fig. 16 indicates that the modulus versus the confining stress relationship can be formulated as

$$G \propto (\sigma_c')^m$$

Fig. 18. Power, $m$, vs. log$\gamma$ relationships compared with those of other researchers

![Fig. 18](image)

where $m$ denotes the slope of each line. As the shear strain, $\gamma$, increases, the slope or $m$ increases. The variation of $m$ along with the increasing strain is plotted on the semi-logarithmic graph in Fig. 18 and compared with analogous relations obtained by other researchers. In most investigations including this one the power $m$ is found to be 0.5 for $\gamma = 1 \times 10^{-4}$, and consistently increases as the strain increases. In this research $m$ is evaluated as 0.47 to 0.84 in the strain interval of $10^{-6}$ to $3 \times 10^{-3}$, whereas according to Iwasaki et al. $m$ shows a wider variation of 0.38 to 0.97 within the same strain range. It should be further noted here that $m$-value given by this investigation for the strain smaller than $\gamma = 10^{-4}$ may be approximated to be 0.5. This allows the establishment of a simplified formulation of the modulus with regard to the confining
stress as \( G_0 \propto (\sigma_0')^{1/2} \) which is a familiar concept for most engineers, and for instance formulated as Eq. (6).

In Fig. 19 the strain-dependency curves of the shear modulus obtained in this research are compared with those proposed by two other investigators using the torsional shear device. The figure first shows that there exists an appreciable difference between the two kinds of curves given by Hardin and Drnevich (1970) and by Iwasaki et al. (1976) both corresponding to the torsional shear tests. The curves of the triaxial tests obtained in this investigation have similar patterns to those of Iwasaki et al. although their locations with respect to \( \gamma \)-axis are shifted slightly to the right compared to those of the same researchers. This discrepancy may have been caused by the difference of the modulus in the small strain range evaluated in the two experiments as indicated in Fig. 17.

![Graph](image)

Fig. 20. \( h \) vs. \( \log \gamma \) relationships compared with those of other researchers

In Fig. 20 the strain-dependent variations of the damping ratio measured in this research are compared with those given by other workers for sands employing other testing devices. It is remarkable that different workers have come up with quite different damping ratios, implying technical difficulties for measuring a reliable damping ratio although the differences will give a considerable effect on the evaluation of dynamic response of vibration analyses. This research noticeably yields the smallest damping ratio of all. It is also obvious that in all the cyclic tests only this test yields the damping ratio for the strain smaller than \( \gamma = 10^{-4} \) which shows a striking coincidence with that measured by Iwasaki et al. using the resonant column device. The existence of mechanical frictions in any cyclic loading test will quite probably lead to the apparent increase of the hysteretic damping. Since the measurement of stress and strain in this triaxial device is completely free from any kind of mechanical frictions, the accuracy of the damping ratio measured in this research should be strongly maintained.

**Poisson’s Ratio of Sand for Small Strain Level**

It is understood that theoretically the shear modulus, \( G \), is the material constant with regard to the effective stress, hence does not change its magnitude in accordance with the drainage condition as long as the dilatancy effect is negligible. By combining \( G \) obtained in the undrained test and the modulus of deformation, \( E \), obtained in the drained test on the same saturated sand sample, Poisson’s ratio, \( \nu \), of sand with respect to the effective stress can easily be derived using the following equation.
Fig. 21. $\bar{\nu}$ vs. log $\gamma$ relationships for small strain level with different confining pressures

$\nu = \frac{E}{2G} - 1$  \hspace{1cm} (9)

With this purpose in mind, one additional cycle of the drained loading was given to the sand after ten cycles of the undrained loading only when the strain level was smaller than $\gamma = 10^{-4}$. Thus $G$ at tenth cycle and $E$ at eleventh cycle were used in Eq. (9) to yield Poisson's ratio since the change of soil properties between $N=10$ and $N=11$ can be ignored for the small strain level. Fig. 21 shows the representative graph of Poisson's ratio versus shear strain under different confining pressures for dense Toyoura sand. It is evident from the graph that the measured Poisson's ratios fall in the narrow band between the two lines slightly ascending with the strain increase. With the confining pressure decreasing, Poisson's ratio appears to increase slightly. In Fig. 22 Poisson's ratios determined as the average of the measured values for the strain smaller than $\gamma = 4 \times 10^{-5}$ are taken against the confining stresses for the sand with different void ratios. It is obviously seen from the graph that Poisson's ratio of clean sand is 0.2 to 0.3 for the small strain range, and that the increase of void ratio as well as the decrease of confining stress contributes to the increase of the dynamic Poisson's ratio.

SUMMARY AND CONCLUSIONS

In this experimental study the cyclic triaxial tests were conducted for the laboratory measurement of dynamic soil properties corresponding to the wide strain range of $10^{-6}$ to $10^{-3}$. For this purpose the instrumentations for measuring stress and strain of the soil specimen in the triaxial chamber were extensively improved so that the measured values are completely free from any mechanical frictions even for the very small strain range of $10^{-5}$ or less. A series of tests for saturated Toyoura sand with various void ratios and confining pressures by this testing method have revealed the following principal facts.

(i) By comparing the two moduli, one obtained with the conventional triaxial device and the other with the improved device, the former modulus tends to diverge as shear strain becomes smaller, while the latter converges to some asymptotic value.
almost equivalent to the insitu shear wave velocity.

(ii) An essentially good agreement of the modulus can be recognized in quantitative manner between the triaxial and the torsional shear test, implying that the different stress and confining conditions inherent to the individual testing devices give no significant effect on the measurement of the modulus.

(iii) The shear modulus, $G_s$, measured in the test for shear strain $\gamma = 10^{-6}$ is almost proportional to the square root of the effective confining pressure, $\sigma_v'$, and formulated in terms of void ratio, $e$, as Eq. (7) which gives larger modulus by 20 percent than the equation of Richart et al. [Eq. (6)].

(iv) The strain-dependency curves of the modulus given in this research are quite reasonable as compared to those proposed by other researchers using the torsional shear device.

(v) The hysteresis damping ratio measured in this research is considerably smaller for all strain range than that obtained by low frequency cyclic tests using the torsional or simple shear devices, although it coincides well with the results of the resonant column tests for the strain range smaller than $\gamma = 10^{-4}$. It is noteworthy that the improved triaxial test is free from any mechanical friction; the factor which possibly leads to an apparent increase of the measured damping ratio, hence the reliability of the damping ratio obtained in this research should be strongly maintained.

(vi) Drained cyclic triaxial tests easier to conduct than undrained tests can give an almost analogous strain-dependent variation of the modulus ratio and the damping ratio to those of undrained tests.

Finally it can be concluded that the improved cyclic triaxial test can be an efficient and dependable method for measuring reliable dynamic soil properties for the wide strain range of $10^{-6}$ to $10^{-3}$ except for the strain level larger than $10^{-3}$, where a special care should be taken since the triaxial stress system discloses its disadvantage due to the unsymmetry of the two alternative stresses of compression and extension. As a direct consequence of the new testing method it is further expected that the static and dynamic soil parameters measured in the triaxial apparatus can be readily connected with the modulus for the small strain level through the use of the improved instrumentation allowing the establishment of experimental relationships between the soil parameters such as shear strength and liquefaction potential and elastic wave velocity. With the accumulation of such laboratory data the insitu wave velocity may serve as a key factor for judging those parameters vital to engineering design.

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NOTATION

$A =$ the triangular areas shown in Fig. 2
$D_{av} =$ mean grain size
$\Delta A =$ the area enclosed in the loop shown in Fig. 2
$\Delta z =$ increment of axial strain
$\Delta \gamma =$ increment of shear strain
\[ \Delta \sigma = \text{increment of axial stress} \]
\[ \Delta \tau = \text{increment of shear stress} \]
\[ e = \text{void ratio} \]
\[ E = \text{modulus of deformation} \]
\[ E_0 = \text{modulus of deformation for } \gamma = 1 \times 10^{-6} \]
\[ G = \text{shear modulus} \]
\[ G_0 = \text{shear modulus for } \gamma = 1 \times 10^{-6} \]
\[ \gamma = \text{specific gravity of grain particles} \]
\[ h = \text{hysteretic damping ratio} \]
\[ m = \text{the power relating } G \text{ and } \sigma_e' \text{ as } G \propto (\sigma_e')^m \]
\[ N = \text{number of cycles} \]
\[ \nu = \text{Poisson's ratio} \]
\[ \rho = \text{Poisson's ratio with respect to effective stress} \]
\[ \sigma_e' = \text{effective confining stress} \]
\[ U_e = \text{uniformity coefficient} \]

REFERENCE


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