RESISTANCE OF PARTLY SATURATED SAND TO LIQUEFACTION WITH REFERENCE TO LONGITUDINAL AND SHEAR WAVE VELOCITIES

YOSHIMICHI TSUKAMOTOⁱ⁾, KENJI ISHIHARAⁱⁱ⁾, HIROSHI NAKAZAWAⁱⁱⁱ⁾,
KUNIO KAMADA^{iv)} and YONGNAN HUANG^{v)}

ABSTRACT

In order to examine the liquefaction resistance of partially saturated sand, a cyclic triaxial test apparatus was manufactured, with which it was possible to first measure the longitudinal and shear wave velocities and then to conduct cyclic triaxial tests on the same specimens under p -constant conditions. A series of the tests indicated that the longitudinal wave (P-wave) velocity could be used to identify the degree of saturation of near-saturated samples, and its effect on the liquefaction resistance could be clarified by conducting cyclic loading tests immediately following measurement of P-wave velocity. The results of the P- and S-wave measurements indicated that the velocity of P-wave propagation, V_p , tends to increase from about 500 m/sec to about 1800 m/sec when the B -value increases from 0 to 0.95 corresponding, respectively, to the saturation ratio S_r of about 90 and 100%. Simple theoretical consideration indicated, on the other hand, that V_p could be expressed as a function of two factors, that is, the B -value and what may be termed "skeleton Poisson's ratio" which is defined for the small-strain level of soil deformation, based on the measured V_p and the propagation velocity, V_s , of the shear wave (S-wave). The theoretically derived simple formula as above was shown to be in good coincidence with the relations obtained in the tests. For the cyclic phase of triaxial tests, the cyclic resistance to liquefaction was shown to increase with a decrease in the B -value and hence with a drop in the saturation ratio S_r . It was also shown that when the B -value drops practically to zero with a V_p -value of about 500 m/sec, the cyclic strength of the specimen becomes twice as much as that at full saturation with $B=0.95$ and $V_p=1800$ m/sec.

Key words: cyclic triaxial test, liquefaction, partial saturation, primary wave velocity, sandy soil (IGC: D7/E8)

INTRODUCTION

Laboratory tests have demonstrated that the resistance of sand to the onset of liquefaction tends to increase with a reduction in the saturation ratio S_r of soil samples. Since the saturation ratio is a quantity requiring measurements of volume of a soil element, it is generally difficult to determine it precisely particularly when the soil element is partly saturated near the state of full saturation. Then, the B -value has been used instead to quantify the state of saturation. As the B -value is defined as the ratio of the induced pore water pressure to the applied confining stress, this value is easily measured in the laboratory and accurate enough to indicate a state of partial saturation. Thus, the B -value has been widely used in laboratory soil testing for evaluating the degree of saturation of soil specimens. According to recent studies by Chaney (1978), Yoshimi et al. (1989) and Grozic et al. (1999, 2000) among others, when the B -value drops to a level of

about zero with a saturation ratio S_r of about 90%, the resistance to liquefaction has been shown to increase roughly two times as much as that of fully saturated samples.

However, a crucial disadvantage of using either the B -value or saturation ratio S_r is that it is practically impossible to monitor these quantities in soil deposits in the field. Then, even if its importance is recognized, there has been no way to monitor the B -value or saturation ratio S_r in any method of field investigations and to duly consider the effects under in-situ conditions in evaluating the liquefaction resistance of in-situ sand deposits.

On the other hand, measurements of propagation velocities of shear wave, V_s , and longitudinal wave, V_p , in the field have been carried out at a number of sites by means of the cross-hole and down-hole techniques, which are now commonly in use in routine investigations. With respect to the velocity of the longitudinal wave (P-wave), field measurements have shown that it often yields values,

ⁱ⁾ Lecturer, Department of Civil Engineering, Tokyo University of Science.

ⁱⁱ⁾ Professor, ditto.

ⁱⁱⁱ⁾ Research Associate, ditto.

^{iv)} Nippon Engineering Consultants Co., Ltd. (Formerly graduate student, Department of Civil Engineering, Tokyo University of Science).

^{v)} Kiso-Jiban Consultants Co., Ltd.

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Fig. 1. Soil profiles at Shinano estuary in Niigata, Japan

which are approximately equal to, or somewhat smaller than, the P-wave velocity travelling through water for the case of saturated loose soil deposits existing at depths below the ground water table. One of such examples of the velocity profile obtained by means of the down-hole method is shown in Fig. 1. This is the soil profile at a site near the mouth of the Shinano River in the city of Niigata, Japan. It may be seen that the velocity of P-wave velocity takes values of $V_p = 1200\text{--}1300$ m/sec down to a depth of 7 m. It is recognized that it is quite common to observe similar velocity profiles in many other cases. This tendency has also been unearthed by Kokusho (2000) in relation to the study of amplification characteristics of longitudinal motions during earthquakes. This fact suggests that the soil deposit several metres below the ground water table is not fully saturated but in a state of near-saturation. Thus, in view of the laboratory-confirmed increased liquefaction resistance of partially but nearly saturated sand as mentioned above, it is highly likely that the in-situ deposits of sands several metres below the ground water table would demonstrate a resistance to liquefaction which is substantially greater than the value hitherto known and used in design practice which assumes the soils to be fully saturated.

There have been few efforts to measure the P-wave velocity in laboratory soil samples thus far because of a lack of recognizing its importance in engineering applications. However, the techniques to monitor it in laboratory tests may be explored without much difficulty. Some efforts have been done in this context recently by Nakagawa et al. (1996, 1997), and by Fioravante (2000), using a set of bender elements attached to triaxial test specimens. Thus, the use of the P-wave velocity has the potential as a means to identify the degree of saturation of soils in field deposits as well as in the laboratory.

The advantage of using P-wave velocity as a liaison parameter for identifying the saturation ratio S_r , and hence the liquefaction resistance, may be summarized as

Fig. 2. Partition of the effective stress $\Delta\sigma'$ and pore water pressure Δu

follows:

(1) P-wave velocity can be measured both in field deposits and in laboratory soil samples while other index properties such as B -value and saturation ratio S_r can by no means be monitored in the field. Therefore, the P-wave velocity may be used as a parameter to identify conditions of laboratory soil samples and in-situ soil deposits in relation to the degree of saturation.

(2) P-wave velocity measured in-situ is considered to possess an equal level of credibility to that monitored in the laboratory, and therefore it could be used to identify the in-situ B -value as it is used for the laboratory soil samples.

On the basis of the conception above, multiple series of laboratory tests were conducted on reconstituted samples of sand with varying saturation ratios. The outcome of these tests is described in the following pages of this paper, along with some theoretical consideration on the Poisson's ratio of porous elastic media.

RELATIONS BETWEEN P-WAVE, S-WAVE VELOCITY AND B -VALUE

An attempt to correlate the B -value with the P- and S-wave velocities has been made by Kokusho (2000) based on the theory of wave propagation through a poroelastic medium. The following is an alternative approach leading to the same formulae, based on Ishihara (1996).

Let it be assumed that the partially saturated soil is composed of the skeleton and the pores containing air as illustrated in Fig. 2. It is assumed that the bubble-like structure of the pores is much more compressible than water itself because of the air dispersed in the water. It is also assumed that the volume of air is small enough so as not to be able to develop any interaction with the soil skeleton such as meniscus, where the degree of saturation is probably below the level of $S_r = 90\%$. Let an overall compressional stress, $\Delta\sigma$, be applied to an element of the partly saturated soil. The stress is divided into two parts; the effective stress, $\Delta\sigma'$, transmitted to the soil skeleton and the pore water pressure, Δu , carried by the air-containing water, as schematically illustrated in Fig. 2. Thus, one obtains,

$$\Delta\sigma = \Delta\sigma' + \Delta u \quad (1)$$

Let it be assumed first that the soil skeleton and the bubble structure of pore fluid are deforming independently

without mutual interaction. If the volume of the soil skeleton V_b is compressed by an amount ΔV_b due to an increase in effective stress $\Delta\sigma'$, the following relation is obtained,

$$\frac{\Delta V_b}{V_b} = C_b \Delta\sigma' \quad (2)$$

where C_b is the compressibility of the soil skeleton. By denoting the porosity with n , the volume of the void consisting of water plus air is given by nV_b . Then, if the pore air-water is compressed by an amount ΔV_w due to the pore pressure increase Δu , the following relation is obtained,

$$\frac{\Delta V_w}{nV_b} = C_1 \Delta u \quad (3)$$

where C_1 is the compressibility of the air-bearing water which is apparently larger than the compressibility of water itself. At this stage, let it be assumed that some constraints are imposed on the occurrence of ΔV_b and ΔV_w , in such a way that the volume change in each of the two phases can not occur independently. These constraints are associated with the drainage conditions of pore-filling air-water. If the amount ΔV_w is greater than ΔV_b , water is entering into the pores, and conversely if $\Delta V_w < \Delta V_b$, water is coming out of the pores. These two situations imply the conditions where drainage is taking place. Thus, it is obvious that the undrained condition is imposed by,

$$\Delta V_b = \Delta V_w \quad (4)$$

This is regarded as a kind of a compatibility condition or a constraint required for otherwise an independently deforming two-phase medium. In fact, the condition of Eq. (4) requires that the soil skeleton and the pore bubble structure deform together by the same amount. Introducing Eqs. (2) and (3) into the undrained condition of Eq. (4), and using the relation of Eq. (1), one obtains,

$$\frac{\Delta V_b}{V_b} = \frac{nC_1}{1 + \frac{nC_1}{C_b}} \Delta\sigma \quad (5)$$

If the soil skeleton is viewed as an elastically deforming medium, then the overall volumetric modulus K is derived from Eq. (5) as,

$$K = \frac{1 + \frac{nC_1}{C_b}}{nC_1} \quad (6)$$

It is well known that the pore pressure coefficient, B , introduced by Skempton (1954) is given as the ratio of $\Delta u / \Delta\sigma$ and defined as,

$$B = \frac{1}{1 + \frac{nC_1}{C_b}}, \quad \text{or} \quad C_1 = \frac{1-B}{B} \frac{C_b}{n} \quad (7)$$

Equation (7) is interpreted as a relation between the com-

pressibility of the air-containing pore water, C_1 , and the compressibility of the skeleton, C_b , which is expressed in terms of the B -value.

In the theory of elasticity, it is known that Poisson's ratio, ν , which will be referred to as overall Poisson's ratio, is expressed as a function of the shear modulus G_0 and the volumetric modulus K as follows,

$$\nu = \frac{1}{2} \frac{3K - 2G_0}{3K + G_0} \quad (8)$$

Note that this equation refers to the parameters ν , K and G_0 , which are defined for overall deformation, not for the soil skeleton deformation. The modulus G_0 entering in Eq. (8) is the shear modulus of the skeleton at small strains, which is assumed to be equal to the shear modulus at small strains associated with the overall deformation. Introducing Eq. (7) into Eq. (6), one obtains,

$$K = \frac{1}{nC_1 B} = \frac{1}{(1-B)C_b} \quad (9)$$

It is thus known that the volumetric modulus of overall deformation, K , is related to that of the soil skeleton, $K_b = 1/C_b$, through the B -value as indicated by Eq. (9). With reference to Eq. (9), Eq. (8) is rewritten as,

$$\nu = \frac{1}{2} \frac{3 - 2G_0 n C_1 B}{3 + G_0 n C_1 B} = \frac{1}{2} \frac{3 - 2G_0 C_b (1-B)}{3 + G_0 C_b (1-B)} \quad (10)$$

It is to be noted that the overall Poisson's ratio, ν , for the overall deformation is expressed in terms of the soil skeleton compressibility C_b , shear modulus of the soil skeleton G_0 and the B -value. On the other hand, according to the theory of elastic wave propagation, the overall Poisson's ratio ν , velocity of S-wave propagation V_s and P-wave propagation V_p are related as follows,

$$\left(\frac{V_p}{V_s} \right)^2 = \frac{2(1-\nu)}{1-2\nu} \quad (11)$$

Introducing Eq. (10) into Eq. (11), one obtains,

$$\left(\frac{V_p}{V_s} \right)^2 = \frac{4}{3} + \frac{1}{G_0 C_b (1-B)} \quad (12)$$

It is to be noted here that the quantity G_0 and C_b are the parameters related to the soil skeleton deformation without any influence of pore water. Thus, the relation of Eq. (12) implies that the ratio of V_p and V_s , pertaining obviously to the overall gross quantities, is correlated with the soil skeleton parameter $G_0 C_b$ and the pore pressure coefficient B , if the undrained condition as expressed by Eq. (4) is assumed to hold valid. Since the wave propagation through near-saturated soils takes place in a sufficiently short period of time without allowing any drainage of pore water, the correlation of Eq. (12) is considered valid and therefore taken as the basic relation to be utilized when attempting to evaluate the B -value from measurements of V_p and V_s , if the value of $G_0 C_b$ is known.

To understand the above concept more visibly, it might be more convenient to express $G_0 C_b$ in terms of Poisson's

Fig. 3. Cross sections of the cap, samples and pedestal with transducers

ratio of the soil skeleton, ν_b , which is defined as follows with reference to the definition of Eq. (8),

$$\nu_b = \frac{1}{2} \frac{3K_b - 2G_0}{3K_b + G_0} = \frac{1}{2} \frac{3 - 2G_0C_b}{3 + G_0C_b}, \quad C_b = \frac{1}{K_b} \quad (13)$$

where the soil skeleton modulus K_b is converted to the compressibility of the soil skeleton C_b through the relation $C_b = 1/K_b$. By introducing the G_0C_b from Eq. (13) into Eq. (12), one obtains,

$$\left(\frac{V_p}{V_s}\right)^2 = \frac{4}{3} + \frac{2(1 - \nu_b)}{3(1 - 2\nu_b)(1 - B)} \quad (14)$$

This is the relation expressing the ratio V_p/V_s in terms of the skeleton Poisson's ratio ν_b and the B -value. Equation (14) is the same formula as that derived by Kokusho (2000). In the studies described below, the deformation of the soil induced by the wave propagation is considered very small, thereby keeping the strain level within an elastic range. Therefore, the value of ν_b is assumed to take a constant value irrespective of the B -value. It is also possible to derive the relation between the skeleton Poisson's ratio ν_b and overall Poisson's ratio ν . This can be done by introducing G_0C_b from Eq. (13) into Eq. (10) as follows,

$$\nu = \frac{3\nu_b + (1 - 2\nu_b)B}{3 - (1 - 2\nu_b)B} \quad (15)$$

This relation was also derived by Kokusho (2000). In Eq.

(15), it is known that $\nu = \nu_b$, when $B = 0$. This implies that ν_b is the Poisson's ratio when there is no pore pressure buildup. Note that the Poisson's ratio of a dry sand takes a value around 0.25 which is apparently different from ν_b being considered here. It has been known in tests that when the saturation ratio S_r of a soil drops to a value near 90%, the B -value becomes practically equal to zero. Thus, the value of ν_b is to be taken as the Poisson's ratio of near-saturated soils when $S_r \approx 90\%$.

DEVELOPMENT OF TEST APPARATUS

Overview

The cylindrical cell, which accommodates a triaxial specimen of 60 mm in diameter and 120 mm in height, was newly fabricated in this study. Located at the top end of a soil specimen, there is a cap equipped with a porous disk, which houses a triggering system of the longitudinal and shear wave pulses. At the bottom end of the soil specimen is a pedestal, also equipped with a porous disk, and houses receivers of the longitudinal and shear wave pulses generated at the cap and travelling through the soil specimen. A controlling system for the cell pressure was incorporated, which enables p -constant cyclic triaxial tests to be performed, as described below.

Transducers and Associated Equipments for P- and S-Wave Velocity Measurements

Figure 3 shows the cross section of the cap and pedestal, which house the transducers for P- and S-wave velocity measurements. Figure 4 shows the flow of signal transmission and acquisition for wave velocity measurement. The transducer for triggering the impulse of the P-wave adopted in this study is a piezo-electrically driven bolt-clamped ceramic transducer, which is about 15 mm in diameter and 42 mm in length. In order for the surface of this transducer to come in contact with triaxial specimens, the transducer is plugged into the cap, in which the base of the transducer rests on the reaction plate and the circumference of the transducer is loosely fixed to the inner surface of the cap with synthetic resin. This transducer is capable of generating impulsive excitation of up to 60 kHz to trigger P-waves whose direction of vibration is in parallel to the direction of wave propagation. However, only a single impulse excitation was employed in this study. It is difficult, therefore, to identify exact values of frequency at which the tests were performed. The transducer for receiving the P-wave is a piezo-electric accelerometer, which is about 17.5 mm in diameter and 9.8 mm in length. This transducer is mechanically fixed to the base of a dummy dualuminium-made block. This dummy block is then plugged into a pedestal of a diameter of 65 mm, and comes in contact with the triaxial specimens, in which the circumference of the dummy block is loosely fixed to the inner surface of the pedestal with synthetic resin. A pair of bender elements were used to trigger and to receive the shear wave impulse which travels through the triaxial specimens. This device has been widely used for laboratory testing (Viggiani and At-

Fig. 4. Flow of signal transmission and acquisition for wave velocity measurement

kinson, 1995). The bender element is a piezo-electrically driven rectangular ceramic tip, which can be excited by applying voltage input and gain voltage output. The end part of each bender element is fixed into the surface of the cap and pedestal. The voltage input is applied to the bender element fixed to the cap, which swings back and forth and causes the shear wave whose direction of vibration is perpendicular to the direction of wave propagation. The bender element fixed to the pedestal is excited upon arrival of the shear wave. The velocity of the P- as well as S-wave can be calculated by measuring the time difference between the time of transmission and the time of arrival.

Cyclic Triaxial Test Device under p -Constant Condition

When partly saturated sand is subjected to cyclic stress, pore water pressure will build up not only from application of shear stress, but also from repetition of purely compressional stress. Therefore, to properly evaluate the undrained response of partly saturated soils by using triaxial tests, it is desirable to conduct tests maintaining the mean principal stress p at a constant value during undrained shear stress application. The advantage of using this testing method can be illustrated, as follows, by considering the pore pressure coefficients A and B (Fredlund and Rahardjo, 1993).

The stress change induced under the triaxial test conditions is assumed to comprise two stress components, $\Delta\sigma_3$ and $\Delta\sigma_1 - \Delta\sigma_3$. The first component represents an increment of the confining stress, and the second one represents an increment of the deviatoric stress. The pore water pressure induced in triaxial samples consists of the two components derived from the above two stress components as follows,

$$\Delta u = B \{ \Delta\sigma_3 + A \Delta(\sigma_1 - \sigma_3) \} \quad (16)$$

where $B \cong 1$ at full saturation and $B < 1$ at partial saturation. Rearranging the above equation leads to the following equation,

$$\begin{aligned} \Delta u &= B \left\{ \Delta \left(\frac{\sigma_1 + 2\sigma_3}{3} \right) + \frac{3A - 1}{3} \Delta(\sigma_1 - \sigma_3) \right\} \\ &= B \left(\Delta p + \frac{3A - 1}{3} \Delta q \right) \end{aligned} \quad (17)$$

At full saturation satisfying $B = 1$, the coefficient A for the deviatoric stress increment is the sole determinant of the pore pressure development. However, at partial saturation with $B < 1$, the coefficients A and B are both involved. When the test is conducted under the p -constant condition, the term $B \Delta p$ in Eq. (16) disappears and the B -value then remains the same during the application of cyclic loads. Thus, the influence of pore pressure buildup purely due to compressional stress can be deleted, permitting the pore pressure due merely to shear stress to be singled out in the laboratory tests. To achieve this goal, an air pressure controlling system was newly fabricated to enable p -constant cyclic triaxial tests to be conducted in this study.

Testing Procedures

Preliminary tests were carried out on Toyoura sand, which is widely used in geotechnical laboratory testing in Japan and is commercially available. This sand is classified as poorly graded clean fine sand with no fines, and its specific gravity is $G_s = 2.65$ with a mean particle diameter $D_{50} = 0.19$ mm, uniformity coefficient of $U_c = 1.70$, and maximum and minimum void ratios $e_{\max} = 0.988$, and $e_{\min} = 0.616$, respectively.

Soil specimens of 60 mm in diameter and 120 mm in height were prepared by the method of air pluviation to

Fig. 5. Records of wave transmission through the sample

achieve the relative density of $D_r = 30$ through 70%. The soil specimens were isotropically consolidated to a confining stress σ'_0 of 98 kPa, and the B -value was measured. It is known that the B -value increases with increasing back pressure. However, the degree of the B -value increase depends largely on the nature of water used in the tests. In this study, when a high B -value is required, de-aired water was used to saturate the specimen and, at a back pressure of 100 to 200 kPa, a B -value in excess of 0.9 was achieved. When tests were done with the B -value ranging between 0 and 0.5, tap water was used with a back pressure of 200–400 kPa. However, it was difficult to achieve the B -value between 0.6 and 0.9 in the test series described in this study. It is of interest for further study to examine a testing method which can achieve such B -values and to measure the velocities of wave propagation under such B -values. The measurements of the velocities of the P- and S-wave propagation were then carried out. Figure 5 shows an example of time records of wave transmission and acquisition obtained in this study. After finishing non-destructive wave velocity measurements, the p -constant cyclic triaxial tests were conducted with the designated cyclic stress ratio $\sigma_d/(2\sigma'_0)$ and sinusoidal cycles of 0.1 Hz, where σ_d is the amplitude of the cyclic

axial stress, and σ'_0 is the effective confining stress at the time of consolidation.

RESULTS OF TESTS OF P- AND S-WAVE VELOCITY MEASUREMENTS

The results of the P- and S-wave velocity measurements on samples of Toyoura sand prepared to a relative density of $D_r = 30\%$ are displayed in Fig. 6(a), in terms of V_s and V_p values plotted versus the B -value. The outcome of the tests shown in Fig. 6(a) indicates that the velocity of S-wave propagation takes a value of $V_s = 212$ m/sec and remains unchanged with an increase in the B -value. It is to be noted that the smallest B -value was actually $B = 0.05$, where the corresponding saturation ratio was about $S_r = 90\%$, and the largest B -value achieved in the tests was $B = 0.95$, with $S_r = 100\%$.

The velocity of propagation of P-wave V_p is also shown in Fig. 6(a) where it is seen that the value of V_p tends to increase significantly with an increase in the B -value from a value of about $V_p = 500$ m/sec at $B = 0$ to $V_p = 1700$ m/sec at $B = 0.95$. Shown also in Fig. 6(a) is the propagation velocity of a compressional wave through water $V_w = 1492$ m/sec that is achieved at the temperature of 20°C. In the case of longitudinal wave propagation through fully saturated soils, the stiffness of soils comes into effect to some extent because of lateral constraint being imposed on the deformation of the soils during the wave propagation. Therefore, the P-wave through fully saturated soils was shown to propagate with a velocity which is 10–15% faster than the velocity through water (Ishihara, 1971). The result of P-wave measurement through the same sand as above at full saturation with $B = 0.95$ is coincident with the outcome of the earlier study.

With reference to the relation of Eq. (12), the P-wave velocity of propagation is known to be expressed as a function of the shear modulus G_0 and the compressibility of the soil skeleton C_b . As C_b is simply a reciprocal of the volumetric modulus K_b , Eq. (12) implies the fact that the ratio of the P-wave and S-wave propagation velocity is expressed as a function of the ratio of volumetric and shear modulus of the soil skeleton, G_0/K_b , and the B -value. It is to be remembered that the ratio G_0/K_b reflects the deformation characteristics of the soil skeleton at small strains. Thus, it was considered to be more physically visible, if the ratio G_0/K_b were expressed in terms of the skeleton Poisson's ratio ν_b , which can be defined by Eq. (13). Thus, with the use of ν_b , the relation of Eq. (12) was converted to Eq. (14).

To provide a reasonable basis for interpreting the results of the tests, the values of V_p computed by Eq. (14) are shown superimposed in Fig. 6(a) versus the B -values for various postulated values of the skeleton Poisson's ratio. Looking over the data points, it would be reasonable to assume that the skeleton Poisson's ratio is $\nu_b = 0.35$.

Based on the measured values of V_p and V_s , the overall Poisson's ratio, ν , was determined through Eq. (11) and these values are shown in Fig. 6(b) in terms of the plot

Fig. 6. P-wave and S-wave velocities versus B -value ($D_r = 30\%$)

versus the measured B -value. Also shown superimposed is a set of theoretical curves from Eq. (15) for various values of ν_b . The comparison of the test data with those obtained from Eq. (15) indicates as well that the dependency of the skeleton Poisson's ratio ν_b on the B -value is unclear; however, the value of $\nu_b = 0.35$ could be taken as a reasonable average value representing the various overall trend of variation of ν with the B -value.

The test data on the P- and S-wave propagation shown in Fig. 6(a) is alternatively displayed in Fig. 6(c), where the ratio V_p/V_s for individual test data is plotted versus the B -value. The plot in this format would be more reasonable to provide a comparison with the theoretically derived curves. Thus, the relation of Eq. (14) is also shown plotted in Fig. 6(c) for comparison sake. It may be seen as well that the value of $\nu_b = 0.35$ could represent, with a reasonable degree of accuracy, the overall feature of variation of V_p/V_s with the B -value.

Similar sets of test data on Toyoura sand for the relative density of $D_r = 40, 50, 60$ and 70% , under otherwise identical conditions, are presented in Figs. 7, 8, 9 and 10, respectively. Looking over the whole sets of test data shown in Figs. 7 through 10 in light of theoretical considerations, it may be concluded that the relation between V_p or V_p/V_s and B -value for Toyoura sand can be represented reasonably well by the theoretical relation of Eq. (14), irrespective of the relative density, if the skeleton Poisson ratio is chosen as $\nu_b = 0.35$. It is of interest to note that the skeleton Poisson's ratio ν_b remains the same independent of the relative density. Other series of similar tests performed under varying confining stresses have shown that the skeleton Poisson's ratio also remains the same irrespective of the confining stress at the time of the consolidation.

RESULTS OF CYCLIC LOADING TESTS

After performing measurements of P-wave and S-wave velocities, the samples with different B -values were subjected to a series of cyclic axial stresses with constant amplitudes under undrained conditions until they deformed to the double-amplitude (DA) axial strain of 5% . The amplitude of the cyclic axial stress, σ_d , divided by twice the confining stress, $2\sigma'_o$, which was necessary to induce 5% DA axial strain is plotted in Figs. 11 and 12 versus the number of cycles N_c for the case of the samples prepared at a relative density of $D_r = 40\%$ and 60% , respectively. It becomes obvious from these figures that the cyclic stress ratio needed to induce 5% DA axial strain tends to increase significantly with decreasing degree of saturation as represented by the B -value. The cyclic stress ratio inducing the 5% DA axial strain in 20 cycles of load application is read off from Figs. 11 and 12 and plotted versus the B -value in Fig. 13. It is apparent in these figures that the cyclic strength as defined above tends to increase with decreasing B -value, and hence with decreasing saturation ratio S_r to a value of about 90% . The similarly defined cyclic strength is presented in Fig. 14 as a function of the P-wave velocity V_p measured at the start of each of the

Fig. 7. P-wave and S-wave velocities versus B -value ($D_r = 40\%$) Fig. 8. P-wave and S-wave velocities versus B -value ($D_r = 50\%$)

Fig. 9. P-wave and S-wave velocities versus B -value ($D_r = 60\%$) Fig. 10. P-wave and S-wave velocities versus B -value ($D_r = 70\%$)

Fig. 11. Cyclic stress ratio versus number of cycles causing 5% DA axial strain ($D_r = 40\%$)

Fig. 12. Cyclic stress ratio versus number of cycles causing 5% DA axial strain ($D_r = 60\%$)

cyclic loading tests, where it is seen that the cyclic resistance tends to increase significantly with a decrease in V_p , particularly when the P-wave velocity becomes less than 500 m/sec and where the B -value drops to less than 0.1 with a saturation ratio S_r of 90%. The same data set is alternatively shown in Fig. 15 as a function of V_p/V_s

Fig. 13. Cyclic strength versus B -value

Fig. 14. Cyclic strength versus P-wave velocity V_p

where it is seen that the cyclic strength begins to increase sharply when the velocity ratio V_p/V_s drops to a value of about 3. The cyclic strength defined above was normalized to a value at full saturation with the B -value of 0.95. The normalized cyclic strength is displayed in Figs. 16 and 17 as a function of V_p and V_p/V_s , respectively. It may be seen in these figures that the cyclic strength becomes larger as the V_p or V_p/V_s decreases and could take a value about twice as much as the cyclic strength at full saturation, when the V_p -value drops to 400 m/sec, or when the velocity ratio V_p/V_s becomes equal to about 2.0. Note that at the state of $V_p/V_s = 2.0$, the B -value is practically equal to zero, but the saturation ratio still remains as high as about $S_r = 90\%$.

In order to examine the general tendency, the data of

Fig. 15. Cyclic strength versus V_p/V_s Fig. 17. Normalized cyclic strength versus the ratio of velocities V_p/V_s Fig. 16. Normalized cyclic strength versus P-wave velocity V_p

Fig. 18. Summary plots of the test data on the normalized cyclic strength of sands

the similar tests performed previously on Toyoura sand (Ishihara et al., 1998), were collected and shown in Fig. 18 together with the data from the present study. Note that the test data in the previous studies were obtained using another test apparatus. The summary plot shown in Fig. 18 indicates that the cyclic strength of partly saturated Toyoura sand tends to increase with decreasing B -value; also, that when it drops to zero with the saturation ratio S_r of about 90%, the cyclic strength reaches a value twice as much as that at full saturation.

CONCLUSIONS

Studies have shown that decreasing degree of satura-

tion tends to increase the resistance to liquefaction of sands. While the degree of saturation can be expressed in terms of the saturation ratio S_r or B -value in the laboratory specimens, it is practically impossible to precisely monitor these quantities in field deposits at a state of near-saturation, where the liquefaction resistance is known to vary significantly. In view of this, P-wave velocity, V_p , was suggested as an alternative parameter which could be used for quantifying the degree of saturation. The usage of P-wave has several advantages, namely

it can be monitored both in-situ and in the laboratory, and it is precise enough to correlate a meaningful change in the saturation ratio S_r or B -value with the cyclic resistance of partly saturated sand.

With these views in mind, an additional setup was attached to the conventional type of cyclic triaxial test apparatus so that the velocities of both P- and S-waves could be monitored in the laboratory samples prior to the cyclic loading tests. The first attempt of this kind was made by installing impact-applying devices and accelerometers to the cyclic triaxial test apparatus (Ishihara et al., 1998). The second attempt of the same context was subsequently made by installing a pair of bender elements in the cyclic triaxial test apparatus at Tokyo University of Science. The results of the test in the second attempt are reported in this paper. First of all, the outcome of the P- and S-wave measurements were presented together with some theoretical consideration based on the pore-elastic deformation characteristics of two-phase media. It was shown by the analysis that the ratio of P- to S-wave (V_p/V_s) could be expressed as a function of the skeleton Poisson's ratio, ν_b , and the B -value. Since the B -value was measured in the laboratory tests together with V_p and V_s , it was possible to plot V_p/V_s versus the B -value and to determine the skeleton Poisson's ratio so that the theoretical curve would best fit the average of the data points. The results of such comparison disclosed that the dependency of the skeleton Poisson's ratio ν_b on the B -value is unclear; however, the value of $\nu_b = 0.35$ can be taken as a reasonable average value representing the various overall trend of variation of ν with the B -value, irrespective of relative density and confining stress for Toyoura sand.

After measurements of P- and S-wave velocities, cyclic loading tests were performed on the same sand sample and the cyclic stress ratio causing liquefaction was determined. As a result of the tests, the resistance to liquefaction was found to increase with decreasing B -value. When the B -value drops to zero with a saturation ratio of $S_r = 90\%$, the cyclic strength becomes twice as much as that at full saturation with $B = 0.95$. With the two observations above, it may be suggested that it would be possible to estimate the B -value of a sand deposit in the field based on in-situ measured values of P- and S-wave velocities, and hence its liquefaction resistance, if the value of $\nu_b = 0.35$ is assumed to be applicable for the in-situ deposit of the sand.

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NOTATION

n :	porosity of soils
p :	mean principal stress
q :	deviatoric stress
u :	pore water pressure
A :	pore water pressure coefficient
B :	pore water pressure coefficient, Skempton's B -value
C :	compressibility
C_b :	compressibility of soil skeleton
C_f :	compressibility of pore air-water
G_0 :	shear modulus
K :	overall volumetric modulus
K_b :	volumetric modulus of soil skeleton
S_r :	saturation ratio
V_b :	volume of soil skeleton
V_w :	volume of pore air-water
V_p :	velocity of propagation of longitudinal wave
V_s :	velocity of propagation of shear wave
σ'_c :	confining stress
σ_d :	cyclic axial stress
ν :	overall Poisson's ratio
ν_b :	skeleton Poisson's ratio

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