28 Mar 2001, 9:00 am - 9:45 am

Keynote Lecture: Recent Studies on Liquefaction Resistance of Sand-Effect of Saturation

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Proceedings: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics and Symposium in Honor of Professor W.D. Liam Finn
San Diego, California, March 26-31, 2001

KEYNOTE LECTURE

RECENT STUDIES ON LIQUEFACTION RESISTANCE OF SAND – EFFECT OF SATURATION

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ABSTRACT

The problem of sandy soils as to how they behave when they contain air or gas has been recently addressed in relation to evaluation of cyclic resistance during earthquakes. In order to shed some light on this issue, some laboratory tests were conducted on sand samples prepared in the triaxial test apparatus. The outcome of the tests disclosed that the degree of imperfect saturation can be quantified by way of the propagation velocity Vp of compressional wave or P-wave and that the cyclic resistance exhibits significant increase if the velocity Vp drops below 700 m/sec, a value smaller than the propagation velocity through water.

INTRODUCTION

It has been known in the laboratory tests that resistance of sand to onset of liquefaction tends to increase with reduction in saturation ratio of samples which is expressed in terms of the B-value. When the B-value drops to a level of 0.1, the resistance to liquefaction has been shown to increase roughly two times as much as that of fully saturated samples having a B-value greater than 0.95 (Chaney, 1978 and Yoshimi et al. 1989). As the B-value is defined as the ratio of induced pore water pressure to the applied confining stress, this value is easy to be measured in the laboratory samples. Thus, it has been widely used in the laboratory soil testing, for evaluating the degree of saturation of test specimens. However, a crucial disadvantage of using either B-value or saturation ratio 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 in any method of field investigations and to duly consider its effects in their in-situ conditions in evaluating liquefaction resistance of sand deposits.

On the other hand, measurements of propagation velocities of S-wave, V_S, and P-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 in use commonly in routine investigation. With respect to the velocity of the P-wave, field measurements have shown that it often yields values which are approximately equal to or somewhat smaller than the P-wave velocity through water for the case of saturated loose soil deposits existing at depths below the ground water table. One of such examples of velocity profile obtained by way 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 is may be seen that the velocity of longitudinal velocity takes values of $V_p = 1200 - 1300$ m/sec down to a depth of 7 m. It is recognized that it is quite common to observe similar velocity profile in many other cases. This tendency has also been unearthed by Kokusho (2000) in relation to the amplification characteristics of longitudinal motions during earthquakes. This fact suggests that the soil deposit several meters below the ground water table is not fully saturated and 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 meters below the ground water table would mobilize resistance to liquefaction which is substantially greater than the value that has been known and used in the design practice assuming the soils to be fully saturated.

Efforts to measure the P-wave velocity in the laboratory samples have not been made extensively thus far because of lack of its recognized importance in engineering application. However, the techniques to monitor it in the laboratory test 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 the bender elements attached to triaxial test specimens. Thus, the use of the P-wave velocity will have a potential as a means to identify the degree of saturation of soils in the field deposits as well as in the laboratory.

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

(1) P-wave velocity can be measured both in the field deposits and in the laboratory samples while other index properties such as B-value and saturation ratio cannot be monitored by any means in the field. Therefore, the P-wave velocity may be used as a parameter to identify conditions of laboratory specimen and that 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 sample.

On the basis of the conception as above, multiple series of laboratory tests were conducted on reconstituted samples of sand with varying saturation ratio. The outcome of these tests is described in the following pages of this paper.

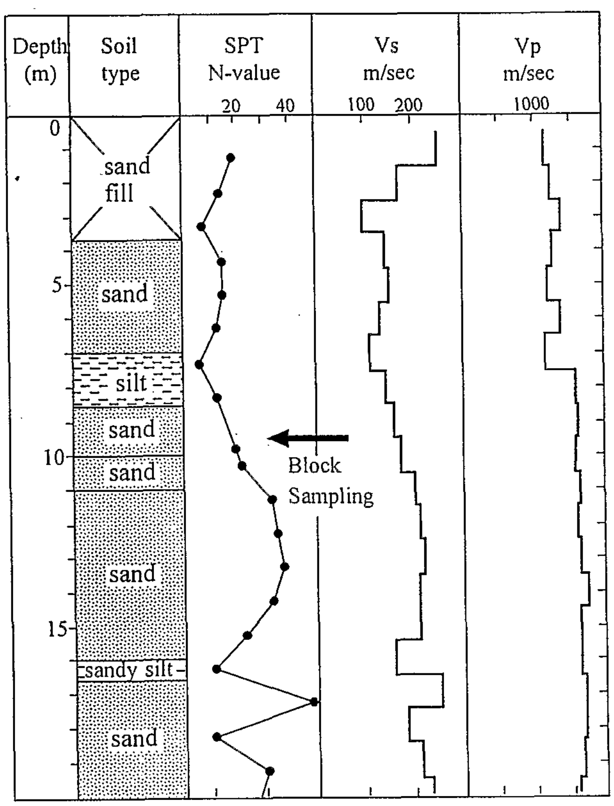

Fig. 1 Soil profile at a site near the mouth of the Shinano River in the city of Niigata

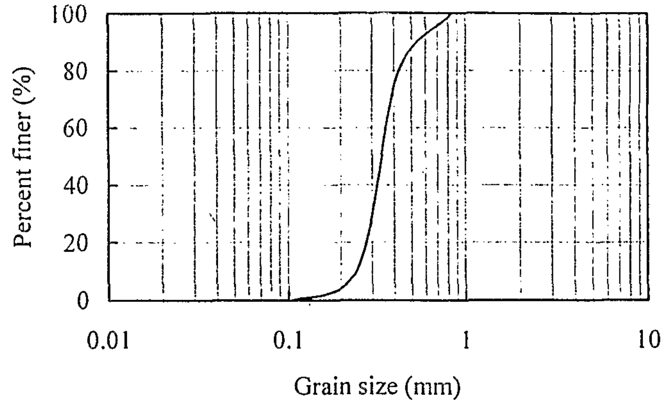

Fig. 2 Grain Size distribution curve of the sand from Niigata used in the test

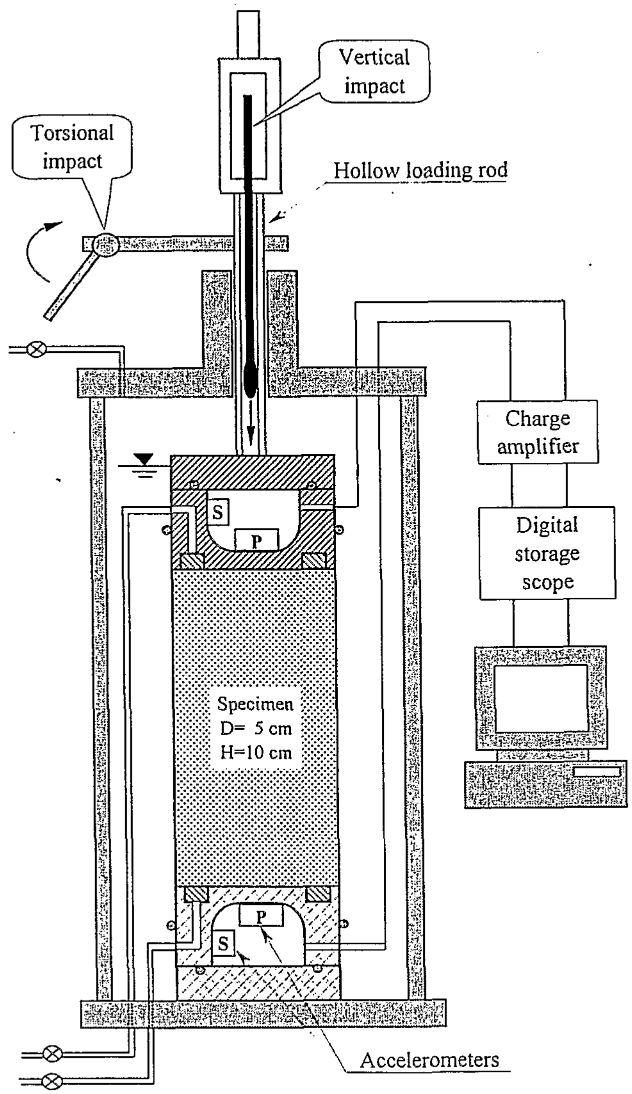

Fig. 3 Test apparatus

TEST SAMPLES AND TEST PERFORMANCE

The excavation had been underway as part of the construction project to provide access roads to the tunnel going across the Shinano River, Niigata, Japan. The soil deposits consist predominately of clean sands as indicated in a soil profile in the vicinity of the sampling site (Fig. 1). The specimens of the sand used in the present study were secured in blocks from the bottom of the excavation at a depth of about 10m from the ground surface. Blocks of soils were secured by cubes 7.5 cm in diameter and 15 cm in height at the exposed surface of intact sand deposits. The grain size distribution curve of the sand is shown in Fig. 2.

The triaxial test apparatus used in the present study is shown in Fig. 3. Two pairs of accelerometers were equipped at the top cap and also at the pedestal of the triaxial apparatus. Vertical and horizontal impact imparted at the top was picked at the bottom of the test specimen as illustrated in Fig. 3.

Specimens of 10cm long and 5cm in diameter were put in place in the triaxial chamber and consolidated to a confining stress of $\sigma'_0 = 118\text{kPa}$. After circulating the de-aired water or tap water, it was possible to put the specimen at a desired B-value or saturation ratio by regulating the back pressure. The tests consisted of two parts. First, a shear wave (S-wave) was generated in the torsional mode by hitting the horizontal arm at the top and by detecting its arrival at the bottom of the specimen. Similarly, compressional wave (P-wave) was generated by applying a vertical impact at the top and by detecting its arrival at the bottom of the specimen. After finishing the first phase of the tests as above, the cyclic axial stress was applied to the specimens until it reached a state of liquefaction with the development of a double-amplitude axial strain of 5 %. Thus, for the same specimen with a known B-value, the velocities of P-wave and S-wave propagation were measured along with the cyclic strength.

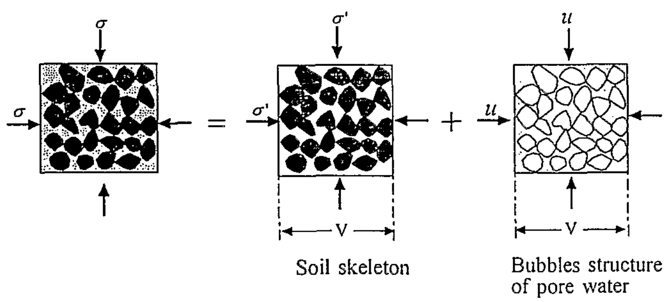

Fig. 4 Partition of the effective stress σ' and pore water pressure u

RELATION BETWEEN P-WAVE VELOCITY AND B-VALUE

The velocity of P-wave propagation is correlated with the B-value through the definition of Poisson's ratio used in the linear theory of elasticity (Ishihara 1996, page 120). Poisson's ratio ν is expressed in terms of the shear modulus G_0 and the volumetric modulus K as

$$\nu = \frac{1}{2} \cdot \frac{3K - 2G_0}{3K + G_0} \tag{1}$$

Let a compressional stress σ be applied to an element of a saturated soil. This stress is divided into two parts: one component, σ', transmitted to soil skeleton and the other, u, carried by water in the pores as schematically illustrated in Fig. 4. Thus, one obtains

$$\sigma = \sigma' + u \tag{2}$$

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 skeleton V is compressed by an amount ΔV due to the effective stress σ', the following relations is obtained:

$$\frac{\Delta V}{V} = C_b \cdot \sigma' \tag{3}$$

where C_b is the compressibility of the soil skeleton. By denoting the porosity by n, the volume of water contained in the bubble structure is given by nV. Then, if the pore water is compressed by an amount ΔV_w due to the pore pressure u, one obtains:

$$\frac{\Delta V_w}{nV} = C_\ell \cdot u \tag{4}$$

where C_ℓ is the compressibility of water itself. The drainage condition is now imposed in terms of relative magnitude of ΔV_w to ΔV. If the amount ΔV_w is grater than ΔV, water is to be taken into the pores, and conversely if $\Delta V_w < \Delta V$, water must come out of the pores. Note that both situations imply drained conditions. Thus it is apparent that the undrained condition is imposed by

$$\Delta V = \Delta V_w \tag{5}$$

This is regarded as a kind of compatibility condition required for independently deforming two-phase medium, that is, the soil skeleton and the bubble structure, to develop a mutual interaction between them.

Introducing Eqs. (3) and (4) into the undrained condition of Eq. (5), and using the relation of Eq. (2), one obtains

$$\frac{\Delta V}{V} = \frac{nC_\ell}{1 + \frac{nC_\ell}{C_b}} \cdot \sigma \tag{6}$$

If the soil skeleton is viewed as an elastically deforming medium, then the volumetric modulus is derived from Eq. (6) as

$$K = \frac{1+\frac{nC_\ell}{Cb}}{nC_\ell} \tag{7}$$

It is well known that the pore pressure coefficient B by Skempton (1954) is given as the ratio of u/σ, therefore, the following relation is obtained.

$$B = \frac{1}{1+\frac{nC_\ell}{Cb}} \tag{8}$$

Introducing this expression into Eq. (7) and further into Eq. (1) one obtains an expression of Poisson's ratio for saturated soils as

$$\nu = \frac{1}{2}\cdot\frac{3-2G_0 nC_\ell B}{3+G_0 nC_\ell B} \tag{9}$$

On the other hand, the relations between the Poisson's ratio, velocities of shear wave propagation V_S and compressional wave propagation V_P are derived as follows from the elastic theory of wave propagation.

$$\left(\frac{V_P}{V_S}\right)^2 = \frac{2(\nu-1)}{2\nu-1} \tag{10}$$

Introducing the relation of Eq. (8) into Eq. (9), one obtains,

$$\left(\frac{V_P}{V_S}\right)^2 = \frac{4}{3}+\frac{1}{G_0 nC_\ell B} \tag{11}$$

For each test specimen in the triaxial cell, the value of porosity n is known and compressibility of water C_ℓ is assumed to take a constant value. Then, if the value of shear modulus G_0 is known for the specimen tested from measurement of the velocity of shear wave propagation V_S, Eq. (10) is regarded as a theoretical relation between the P-wave propagation velocity, V_P, and B-value.

RESULTS OF WAVE VELOCITY MEASUREMENTS

Results of the velocity measurements and the B-value obtained from the tests are shown in Fig. 5 where the measured velocities are plotted versus the B-value. It may be seen that the velocity of shear wave or S-wave propagation V_S remained unchanged irrespective of changes in B-value. However, P-wave velocity V_P is shown to exhibit an appreciable reduction with decreasing B-value from a value of V_P = 1600 m/sec to 400 m/sec. It is apparent that the former value corresponds to the wave velocity through water. The relation of Eq. (11) derived from the theory is also shown in Fig. 5 with a dashed line. It may be seen that the reduction in the measured velocity V_P is not so acute as that of the theory within the B-value range between 0.5 and 1.0. The apparent difference in the measured and theoretically estimated values cannot be identified at present and it is a pending issue yet to be clarified.

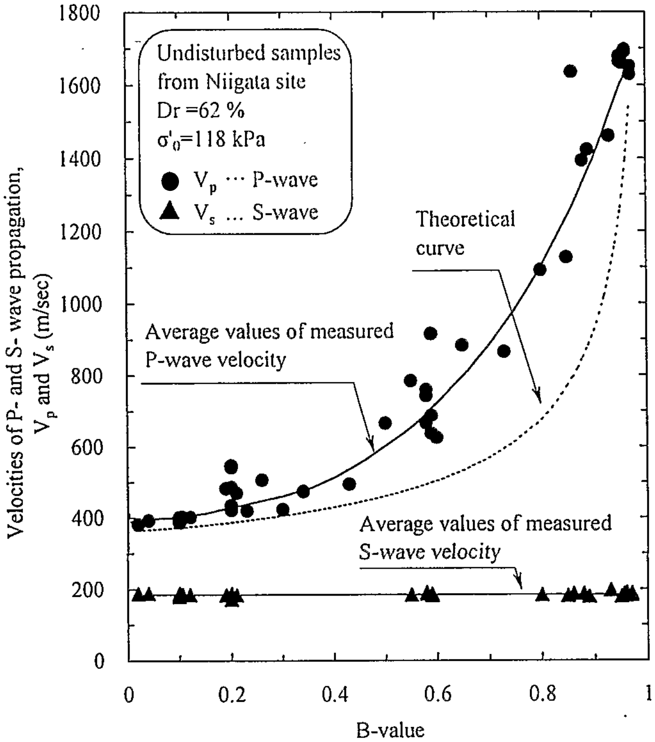

Fig.5 Velocities of P-wave and S-wave propagation versus the B-value

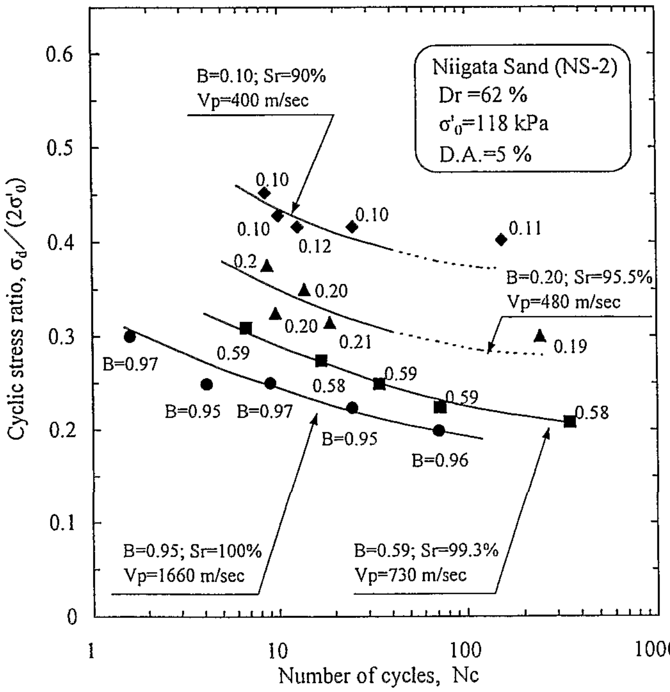

Fig. 6 Cyclic strength versus the number of cycles to cause 5% double-amplitude axial strain

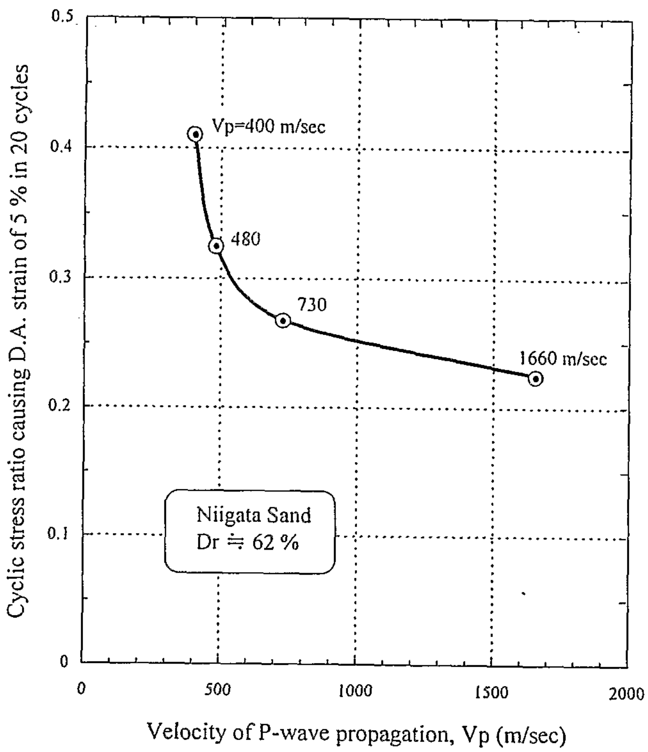

Fig. 7 Cyclic strength of sand versus the velocity of P-wave propagation

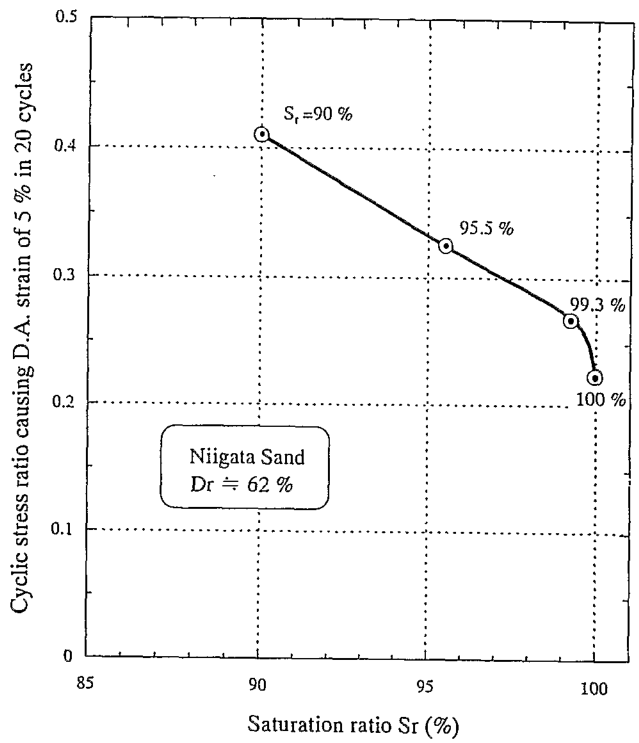

Fig. 9 Cyclic strength versus the saturation ratio

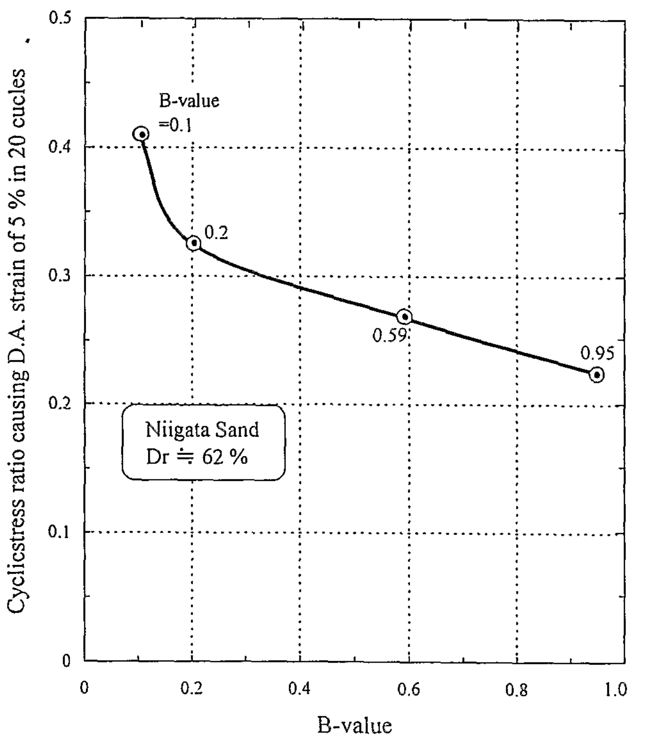

Fig. 8 Cyclic strength versus B-value

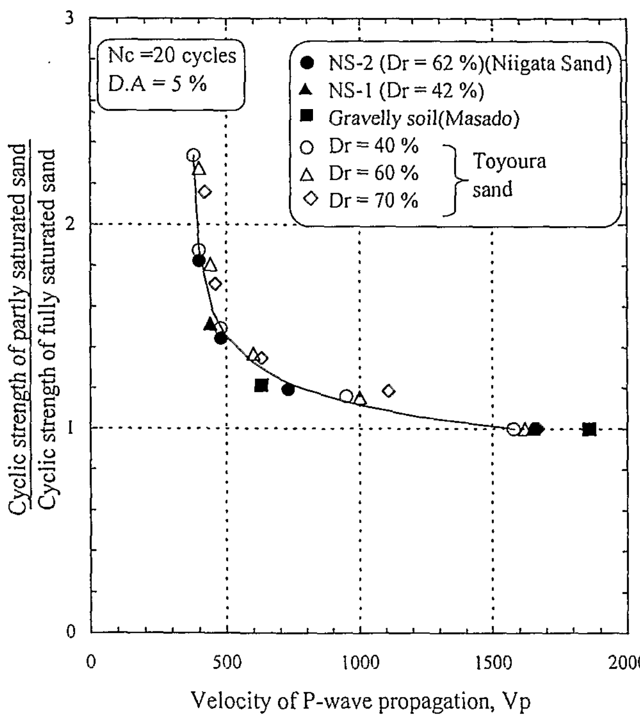

Fig. 10 Ratio of cyclic strength between partially fully saturated sand

RELATION BETWEEN LIQUEFACTION RESISTANCE AND P-WAVE VELOCITY

The results of the cyclic phase loading tests mentioned above are presented in Fig. 6 in terms of the cyclic stress ratio, $\sigma_d/(2\sigma'_o)$, plotted versus the number of cycles required to generate the double-amplitude axial strain (D.A.) of 5 % in the triaxial specimens. As indicated in the figure, the relative density of the undisturbed samples from Niigata site was $D_r = 62$ %. The test data indicate that the cyclic strength defined above tends to increase with decreasing B-value, and hence with decreasing velocity of P-wave propagation through the sample.

The cyclic stress ratio required to induce 5 % D.A. axial strain in 20 cycles of uniform load application was read off from the curves in Fig. 6 and plotted in Fig. 7 versus the velocity of the P-wave propagation. It can be seen that the cyclic stress ratio tends to increase significantly when soils become less and less saturated as quantitatively represented by the decreasing value of P-wave velocity. From the same data file, it is possible to establish a relation between the cyclic strength and B-value as indicated in Fig. 8 where it may be seen that the value of cyclic strength tends to increase significantly when the B-value drops to the level of B=0.1. The saturation ratio, S_r, measured in the test may be used as a parameter to quantify the degree of saturation. In this vein, plot was made in Fig. 9 for the cyclic strength versus the saturation ratio. It may be seen that at the saturation ratio of $S_r = 90$ % corresponding to the B-value of 0.1, the cyclic strength becomes 1.8 times as much as that in the case of $S_r = 100$ %. It should be noticed, however, that with further decrease in S_r below 90 %, the behaviour of sand in expected to become closer to that observed in drained loading condition and the cyclic strength to become conversely smaller particularly when the sand is dense.

To explore the effect of partial and near saturation more in details, the cyclic stress at any P-wave velocity was normalized to the cyclic stress ratio for the fully saturated specimen having a V_P-value through water.

The cyclic stress ratio normalized in this way is plotted in Fig. 10 versus the P-wave velocity V_P. The data obtained preciously in similar tests for Toyoura sand and gravel-containing Masado sand from Kobe are also presented in Fig. 10. It may be seen that all the data lies consistently on a unique line indicating clear tend of increasing cyclic resistance to liquefaction with decreasing P-wave velocity. It is noted that, when the V_P-value is 500 m/sec which could often be the case in field deposits, the cyclic resistance to liquefaction is shown to increase by 50 % as compared to that of the fully saturated soil with a V_P-value of 1600 m/sec. This chart might be utilized to correct the in-situ cyclic strength which is estimated based on the field parameters such as SPT N-value or qc-value in the cone penetration test.

CONCLUSIONS

The P-wave velocity measurements were carried out first on the sample with a prescribed B-value and then the cyclic loading test was performed on the same sample to determine the liquefaction resistance. The results of the laboratory tests disclosed that the cyclic strength increases approximately 1.8 times as the saturation ratio drops from Sr = 100 % down to 96 % with corresponding reduction in B-value from 0.95 to 0.1. The associated reduction of P-wave velocity was shown to be from 1600 m/sec to 400 m/sec. Thus, by utilizing the correlation as established above, the P-wave velocity measured in-situ may be used to correct values of in-situ cyclic strength estimated by the conventional means such as SPT and CPT and to determine true values of cyclic strength of in-situ soil deposits.

ACKNOWLEGEMENTS

The sampling at the site in Niigata city was made possible by the kind assistance of Dr. K. Takashi, Director of the Construction Bureau of Niigata, Ministry of Transport. The authors wish to express their sincere gratitude to him.

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