EXPERIMENTAL STUDY ON RECONSOLIDATION ON RECONSOLIDATION VOLUMETRIC BEHAVIOR OF SAND-GRAVEL COMPOSITES DUE TO DYNAMIC LOADING

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ABSTRACT :

A series of tests are performed to illustrate the reconsolidation volumetric behavior of sand-gravel composites after dynamic loading. The results show that, there is a good correlativity between the reconsolidation volumetric strain, the pore water pressure ratio, and the maximal double amplitude shear strain induced by dynamic loading, though the initial effective consolidation pressure has a great effect on the reconsolidation volumetric strain. The reconsolidation volumetric strain is not affected by the characteristic of dynamic loading and reduces with the increase of specimen relative density. Based on the test results of sand-gravel and sand by Ishihara, an experiential relation of reconsolidation volumetric strain of sand-gravel and sand is suggested. The application method of the experiential relation is given for predicting the ground settlement.

KEYWORDS: reconsolidation volumetric, strain sand-gravel composites, triaxial test

1.INTRODUCTION

Large ground deformation induced by liquefaction due to cyclic loading, such as earthquakes or sea wave loading, can cause huge damages to various structures and lifelines. It has two kinds of expressions, lateral ground displacement and ground surface subsidence. The subsidence of ground surface is mainly caused by the compaction of sand during the dissipation of excess pore water pressure. During a cyclic loading, the excess pore water pressure will increase in saturated sand deposits. Then under the effect of hydraulic gradient, water will seep through sand toward the ground surface. Thus the volume of sand will decrease. Such a kind of volume decrease will cause the ground surface or structures to sub side. Subsidence and differential subsidence will cause damages to lifelines and other kinds of structures. Even the subsidence will cause flood at riverside or seashore. There are many cases of such damages. In the Taiwan earthquake of September 1999, a triangular surveying origin in Nantou County subsided 50 centimeters(Liu, 2003).. In the Niigata earthquake of June 1964, some part of the city was reported to have subsided 0.5m. In the Haicheng earthquake of 1975 and the Tangshan earthquake of 1976, such kinds of damages were found in large areas. There are many other reports about this kind of damage. Nagase and Ishihara (Nagase, H. and Ishihara, 1988) conducted some research about this problem by using a special simple shear test device. The device can apply two horizontal loads perpendicular to each other at the same time. And the dynamic loads used are irregular waves. Nagase drew some useful conclusions which are in agreement with the conclusions of Lee (Lee, K. L. and Albaisa, 1974). Based on the test results, Ishihara and Yoshimine (Ishihara, 1993; Ishihara, 1992) put forward a method to predict the settlement induced by earthquakes. Shamoto (Shamoto, 1996) pointed out that the reconsolidation volumetric strain has good relations with the maximum shear strain induced by dynamic loads. The researches on the effect of effective confining pressure are relatively few (Liu, 2003). In the current research the author finds that the influence of effective confining pressure on reconsolidation volumetric strain can not be neglected under certain conditions.However, Most of these studies aimed at sand since large-scale triaxial apparatus is not available in most laboratories.

As a natural foundation and filled material, sand-gravel composites has the characteristic of low compressibility and high shear strength. Due to its high hydraulic conductivity, gravels and gravelly soils were once thought to be unliquefiable. However, cases have occasionally been reported where liquefaction-associated damage took place in gravelly soil. For instance, at the time of the Haicheng earthquake of February 4, 1975 and Tangshan earthquake of July 28, 1976 in China, signs of disastrous liquefaction were observed in sand-gravel composites filled dam foundation. During the 1983 Borah Peak earthquake in the US, liquefaction was reported to have occurred in gravelly soil deposits at several sites, causing lateral spreading over the gently sloping hillsides. While the drainage conditions surrounding gravelly deposits may exert some influence on the dissipation of pore water pressure and hence on the liquefiability, it is of prime importance to clarify the resistance of gravelly sand itself to cyclic loading. One of the earlier endeavors in this context was made by Wong et al. (1975) who performed a series of cyclic triaxial tests on reconstituted specimens of gravelly soils with different gradation by means of a large-size triaxial test apparatus. The results of these tests indicated somewhat higher cyclic strength as compared to the strength of clean sands. However, whether the result of such tests reflects the cyclic strength of in situ deposits remained open to question. These liquefaction-induced failures in gravel and gravelly soils prompted a critical reevaluation of the behavior of gravelly soils subjected to dynamic loading.

In this study, A series of tests are performed to illustrate the reconsolidation volumetric behavior of sand-gravel composites after dynamic loading. The results show that, there is a good correlativity between the reconsolidation volumetric strain, the pore water pressure ratio, and the maximal double amplitude shear strain induced by dynamic loading, though the initial effective consolidation pressure has a great effect on the reconsolidation volumetric strain. The reconsolidation volumetric strain is not affected by the characteristic of dynamic loading and reduces with the increase of specimen relative density. Based on the test results of sand-gravel and sand by Ishihara, an experiential relation of reconsolidation volumetric strain of sand-gravel and sand is suggested. The application method of the experiential relation is given for predicting the ground settlement.

2.Materials and apparatus

Materials Description and Maximum and Minimum Density Test.

Nierji Dam (Heilongjiang Province, in China) foundation sand-gravel composites was used to investigate liquefaction characteristics in this study. Scalping the oversized particles and taking similar grade method, sand was got for parallel tests. Grain size distributions are shown in Fig. 1. For the triaxial specimen diameter of 200 mm used in this study, the ratio of the specimen diameter to the maximum particle size is about 5. A ratio of 5 to 6 is generally considered necessary for meaningful test results.

To determine the relative density, maximum and minimum dry density tests were performed for the sand-gravel composites and sand first. The maximum density was determined by vibratory compaction method. Specimens were vibrated in a cylindrical mold 100 mm inner diameter and 150 mm high. The minimum density was determined using a cylinder 50mm inner diameter and 400 mm high. The measuring cylinder was filled sand-gravel or sand no more than 1/3 to 1/2, capped with hand, then upset and uprighted carefully 3 to 4 times to achieve a very loose state, and the volume could be got. In Table 1, the characteristics for the density tests and fine content tests are presented.

Table 2.1. Characters of sand-gravel composites and sand tested

Material	Grain size /mm									ρ_{dmax} g/cm^3	ρ_{dmin} g/cm^3	D_{50} /mm	G_s
	>40	40~20	20~10	10~5	5~2	2~1	1~0.25	0.25~0.1	<0.1				
Sand-gravel	0	8.6	28.66	12.94	10.96	8.5	24	5.53	0.77	2.205	1.808	5.1	2.64
Sand	0	0	0	0	19.5	20.6	20.3	22.7	16.9	2.150	1.642	0.53	2.75

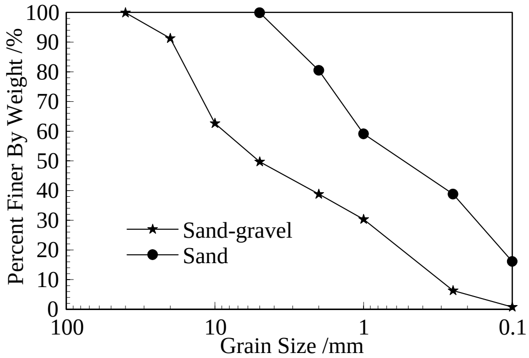


Figure 1 Grain Size Distribution Curves for Tested Materials

Relative density D_r is a pertinent parameter to evaluate undrained cyclic strength of granular soils of different particle gradations and defined by the maximum and minimum dry density ρ_{max} and ρ_{min}, respectively, as Eqn. 2.1.

$$D_r = \frac{1/\rho_{\min} - 1/\rho}{1/\rho_{\min} - 1/\rho_{\max}} \times 100\% \tag{2.1}$$

Here, $\rho_{\max}$ and $\rho_{\min}$ were determined as discussed above, and ρ_d is actual density tested.

Experimental Setup. The axial apparatus used in the tests is illustrated in Fig.2. An inner load transducer was equipped in the chamber, and three deformation transducer, LDT, Gap sensor and external deformation transducer, were used for precise measurements of strain. Gap sensor consists of two small non-contacting discs each encasing an electromagnetic coil. The setup of this type of equipment was developed by Kokusho (1980). As an alternative technique, a device called a local deformation transducer (LDT) was developed by Goto et al. (1991) for measuring small strains of sedimentary rocks tested in the triaxial chamber. A pair of LDTs sit around the specimen shown in Fig. 2. This device is claimed to permit precise measurement of shear strain to be made even to an infinitesimally small strain on the order of 10^{-6}.

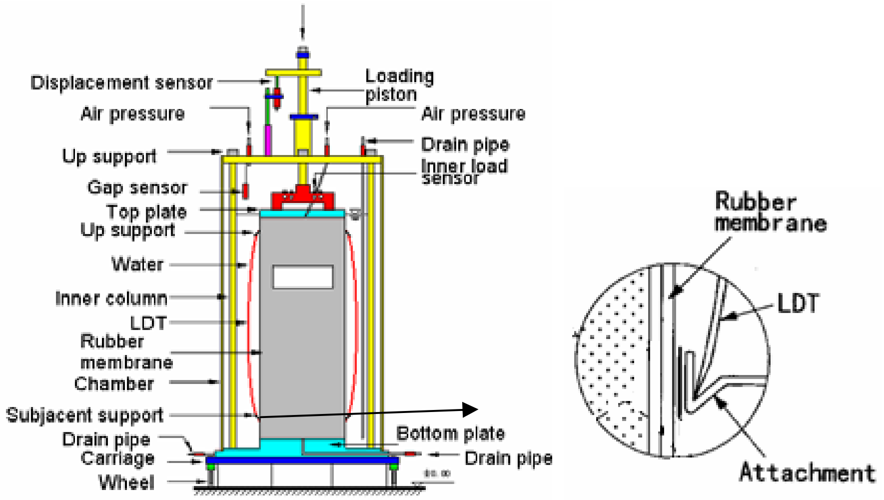


Figure 2 Triaxial test device

Test Method. No special consideration has been given to the effect of membrane penetration is this study, although it is discussed by Ramana and Raju (1981), Vaid and Negussey (1984), Seed et al. (1989), Evans et al. (1992) and Sivathayalan and Vaid (1998). A thicker membrane made of latex, whose thickness was 2 mm, was used for sand-gravel instead of the membrane, also made of latex, having 0.2 mm thickness used for sand. The specimen size of sand-gravel composites was d = 200 mm diameter and h = 500 mm height and that of sand was d = 61.8 mm diameter and h = 125 mm height. All the test procedure is outlined as follows:

1. The specimen was constructed in a mould and carefully tamped in five equal-mass layers 100 mm height by a 10 cm diameter rod in order to control specimen density.
2. The specimen was fully saturated and consolidated isotropically under a certain confining pressure, p_c'.
3. In the undrained cyclic loading triaxial tests, the axial stress was cyclically controlled by sinusoidal waves with frequency of 0.5 Hz based on the fact that undrained strength of noncohesive soils is almost independent of the loading frequency. The cyclic loading was continued until the water pore pressure reached the preconcerted value.
4. After the dynamic loading was stopped, the drain valve was opened and the volume change was measured.

3.Definition of the Factor of Safety for Liquefaction

The basic mechanism of onset of liquefaction is elucidated from observation of the behavior of a sand sample undergoing cyclic stress application in the laboratory triaxial test apparatus. When the axial stress σ_d is applied undrained, the shear stress induced on the 45° plane is $\sigma_d/2$. The normal stress of $\sigma_d/2$ which is mostly transmitted to pore water without inducing any change in the existing effective confining stress σ_0'.

The factor of safety for liquefaction is defined as Eqn. 3.1, the schematic diagram is shown in Fig.3.

$$F_L = \frac{n}{m} \tag{3.1}$$

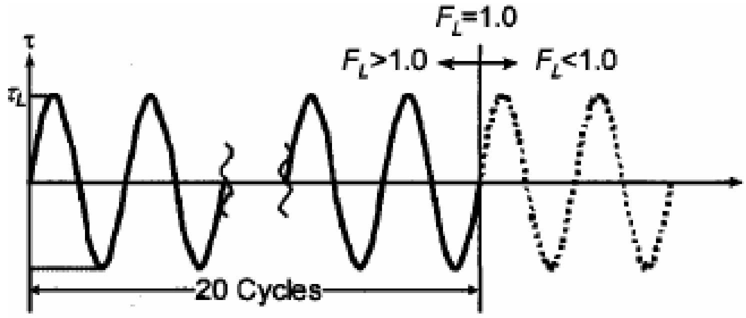


Fig.3 Schematic diagram of F_L definition

Here, m is the times under the determined dynamic loading induced the water pore pressure in the specimens increase to the initial confined pressure, n is the dynamic loading processing times in fact. If the F_L equal to or less than 1.0, then liquefaction is said to take place.

4. Analysis of Tests Results

When saturated sand gravel composites are subjected to shaking during an earthquake, pore water pressure is build up, leading to liquefaction or loss of strength. The pore water pressure then starts to dissipate mainly towards the ground surface, accompanied by some volume change of the com- posites which is manifested on the ground surface as settlements.

The first is the relation of the volume change of sand gravel composites and maximum water pore pressure and shear strains. The outcome of the unconsolidated undrained shear tests after dynamic loading on sand gravel composites with relative density of 50% is shown in Fig.4 and Fig.5. which plots the volumetric strain during the the reconsolidation ε_{vr} against the maximum shear strain γ_{max} experienced by the sample during the undrained regular loading. Wave 1 is a normal sinusoidal wave, Wave 2 is also a sinusoidal wave but the amplitudes of several cycles of the middle part are enlarged. Both of the frequency of wave 1 and wave 2 are 0.5 Hz. It is important to note in Fig.5, when the maximum shear strain increase less than 4%-5%, the volumetric strain during reconsolidation tends to increase significantly.

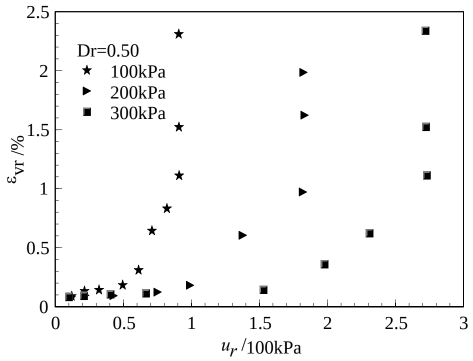


Fig. 4 Relationship between ε_{vr} and u

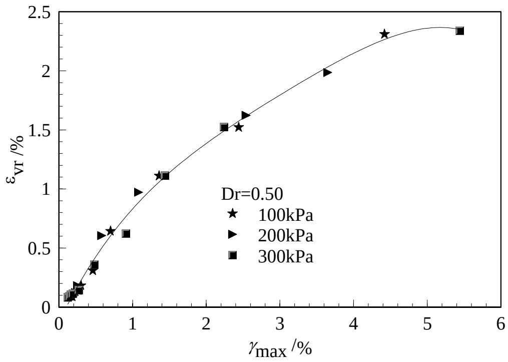
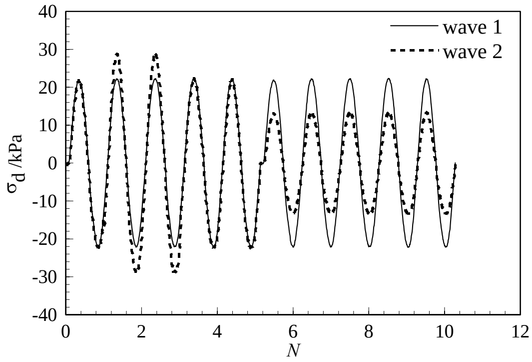


Fig. 5 Relationship between ε_{vr} and γ_{max}



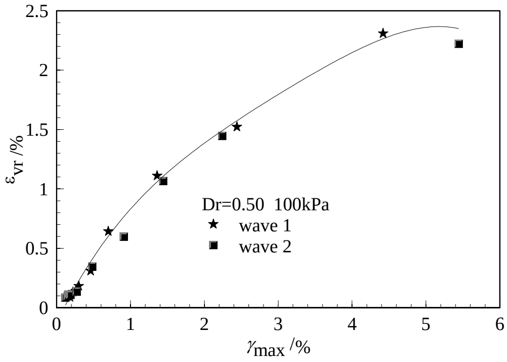


Fig. 6 History curves of different dynamic loads **Fig. 7** Relationship between ε_{vr} and γ_{max}

In order to investigate the influence of the loading properties on the reconsolidated volumetric strain, the irregular loading tests was performed. The loading curve is shown as Fig.5 and the tests results are illustrated as Fig.7. It can be seen that the loading properties have a little influence on the reconsolidated volumetric strain.Tests results of several relative density sand gravel specimens are shown as Fig. 8, and sand specimens tests results by Ishihara are compared together.

In Fig.8, it can be assumed that when the maximum shear strain over than 4%, the reconsolidated volumetric strain will keep stead. So the relationship between the maximum shear strain and the reconsolidated volumetric strain can be represented as Eqn. 4.1.

$$\varepsilon_v = \begin{cases} a\gamma_{max}/4 & \gamma_{max} \le 4 \\ a & \gamma_{max} > 4 \end{cases} \tag{4.1}$$

In order to estimate the liquefaction induced settlement of a sand gravel composites deposit using the correlation shown in Fig.7, it is necessary to know the maximum shear strain that the sand gravel will undergo during the application of dynamic loading in a future earthquake. This should be determined by the concept of the factor of safety for liquefaction. In Fig.10, the correlation of F_L and the maximum shear strain is given, and the clean sand tests results by Ishihara are given also.

The relationship between of the value of a and the relative density D_r can be represented as Eqn. 4.2.

$$a = \alpha \log D_r + \beta \tag{4.2}$$

Based on the test results, it can be got that α=-0.8328, β=0.317. According to Eqn. 4.1 and Eqn. 4.2, the predicted results are shown in Fig.9.

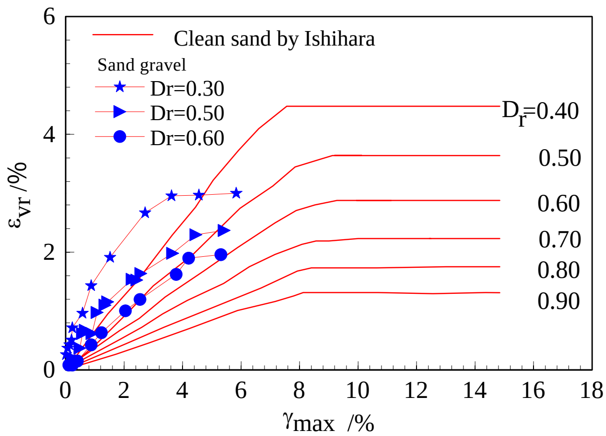


Fig.8 Sand and sand-gravel post-liquefaction volumetric strain plotted against maximum shear strain

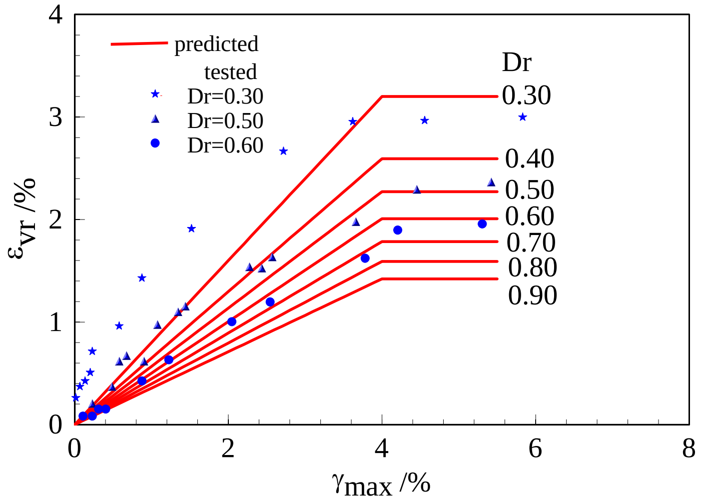


Fig.9 Sand-gravel post-liquefaction volumetric strain plotted against maximum shear strain

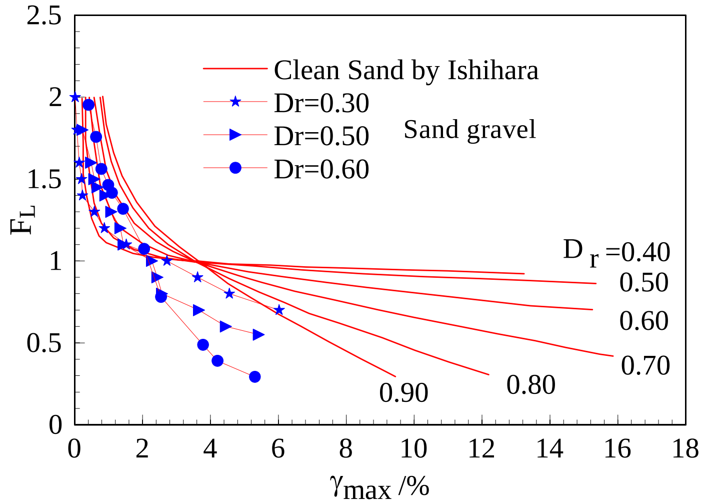


Fig. 10 Relation of factor of safety and maximum shear strain of sand and sand-gravel

Assumed that the relation of factor of safety and maximum shear strain is hyperbola curve, it can be described as Eqn. 4.3.

$$F_L = \frac{a\gamma_{\max}^2 + b\gamma_{\max} + c}{d\gamma_{\max}} \tag{4.3}$$

It can be found that all the test curves of sand gravel specimens via the same point($\gamma_{\max}$=2.5%, F_L=1). Set d=1.0, it can be got that

$$c = 2.5 - 6.25a - 2.5b \tag{4.4}$$

Here, based on the test results, parameter a and b can be presented as:

$$a = 0.281 + 1.017D_r - 1.581D_r^2 \tag{4.5}$$

$$b = 1.404 - 0.908D_r + 1.489D_r^2 \tag{4.6}$$

Then, the maximum shear strain can be expressed as Eqn. 4.7.

$$\gamma_{\max} = \frac{F_L - b - \sqrt{(b - F_L)^2 - 4ac}}{2a} \tag{4.7}$$

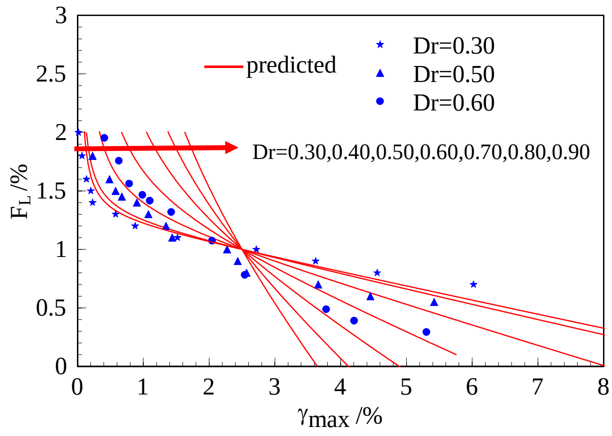


Fig. 11 Relation of factor of safety and maximum shear strain of sand-gravel

With the known factor of safety, according to Eqn. 4.3 to Eqn. 4.7, the predicted results are shown in Fig.11. With the volumetric strains established for each layer throughout the depth of the deposit , the amount of settlement on the ground surface can be calculated by addition of the vertical displacements produced in each layer.

5. Conclusions

The settlement induced by cyclic loading, such as earthquakes or sea waves will do serious damages to various structures. This settlement is caused mainly by the compaction of soil. A series of laboratory tests were conducted on saturated sand gravel composites. It is found that the reconsolidation volumetric strain is correlated with the maximum shear strain before initial liquefaction. No matter whether liquefaction occurs or not, the reconsolidation volumetric strain can be expressed as a function of maximum shear strain, which induced by the dynamic loading.A formula has been proposed for calculation of the reconsolidation volumetric strain, and it can take the effect of the factor of safety for liquefaction into account.

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