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DYNAMIC PROPERTIES OF WEATHERED SOIL DEPOSIT INFLICTED BY THE KOBE EARTHQUAKE

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ABSTRACT

The soil prevailing in the wide area damaged by the 1995 Kobe Earthquake is disintegrated weathered granite. Since the Kobe area has long been deemed seismically inactive, little has been known about the dynamic properties of this type of soil. To provide some basic data for identification of site characteristics, a series of tests have been performed on reconstituted specimens of the soil from the Kobe area using the hollow cylindrical torsional test apparatus. In addition, cyclic tests were performed using uniform loading as well as irregular time histories. This paper discusses the results of these tests as well as the salient features of the behavior of the deposit when subjected to seismic loads.

KEYWORDS

dynamic properties; liquefaction tests; strain dependent; shear modulus; damping; torsional shear tests; reconstituted specimen; cyclic loading; load irregularity; weathered granite

INTRODUCTION

On January 17, 1995, an earthquake of magnitude 7.2 occurred in western Japan, resulting in death of more than 5,000 people and causing widespread damage to buildings and other civil engineering structures. Among the affected areas, the city of Kobe suffered extensive damage as a result of soil liquefaction. Sand boils, ground settlement and lateral spreading were evident in the area, especially on natural and artificial fill deposits along much of the shoreline on the north side of Osaka Bay. Most severely damaged were the areas of landfills in the two large man-made islands, namely, the Port Island and Rokko Island.

Since the Kobe area has long been regarded as seismically inactive, little has been known about the dynamic properties of the soil in the area, such as strain-dependency of modulus and damping characteristics, and liquefaction strengths. This paper intends to describe some of the test results in the laboratory which are of use for the identification of the site characteristics.

SOIL CHARACTERISTICS IN RECLAIMED AREAS

The heavily damaged Port Island and Rokko Island were originally constructed by reclaiming near-shore water 12 to 15 m deep using decomposed granite (locally referred to as Masado) quarried from the nearby Rokko Mountains north of Kobe City during the period of 1967-1981 and 1973-1992, respectively. The reclamation work was carried out by transporting the materials to the fill sites by bottom-barges and loosely dumped over

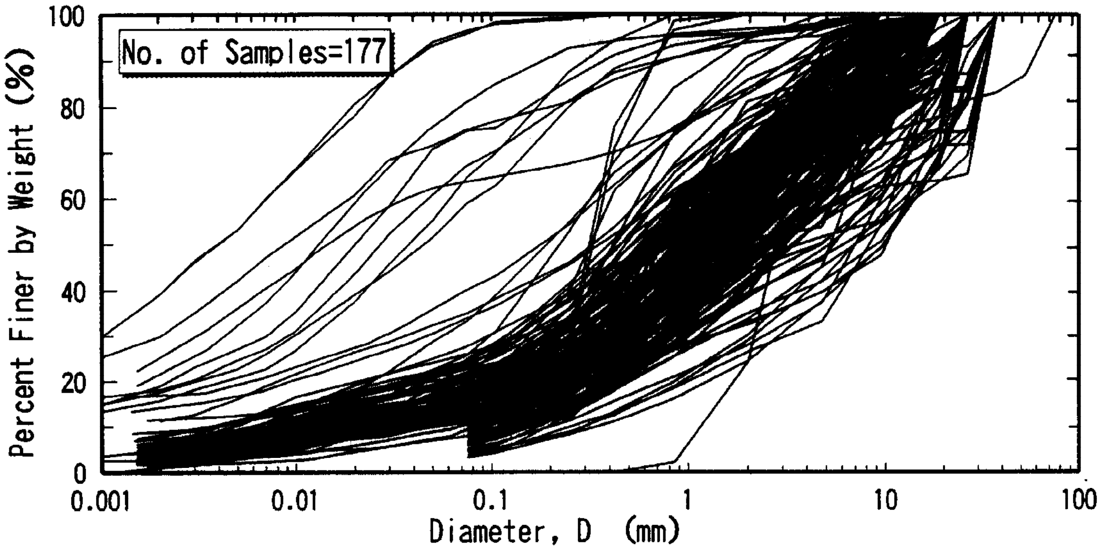

Figure 1: Grain size distribution curves of weathered granite used for landfilling in Port Island

the seabed. The granite-based soil used for the reclamation work contained particles ranging widely in size, from gravel to silt. Figure 1 shows typical grain size distribution curves of the soil used for the reclamation of Port Island. Figure 2, on the other hand, shows a typical soil profile in Port Island, where it can be seen that the reclaimed sand about 14 m thick is underlaid by a layer of soft clay which is the original seabed deposit before the reclamation. Also shown in the figure are V_s and V_p profiles obtained from PS-logging tests.

SHEAR MODULUS AND DAMPING CHARACTERISTICS

In order to provide basic information concerning the shear modulus and damping characteristics of the weathered granite deposit, a series of torsional shear tests were performed on reconstituted hollow cylindrical specimens. The height of the specimen was 70 cm, and the inner and outer diameters were 70 cm and 30 cm, respectively. The samples were prepared by the method of wet tamping. Measurements of shear moduli and damping ratios were performed on specimens which were consolidated anisotropically under two levels of overburden pressures: σ'_v= 100 kPa and σ'_v=50 kPa. For both cases, σ'_v/σ'_h=2.0. For all the tests, the void ratios of the specimens were chosen as more or less constant (i.e., e=0.461~0.481), reflecting the values obtained from undisturbed samples by in-situ freezing. Four tests were performed for each level of confining pressure. In the tests, samples were sheared cyclically ten times, initially at shear strain amplitude of about 5×10^{-6} by controlling the shear stress amplitude. The samples were repeatedly sheared cyclically at increasing strain levels until a shearing failure occurred. The

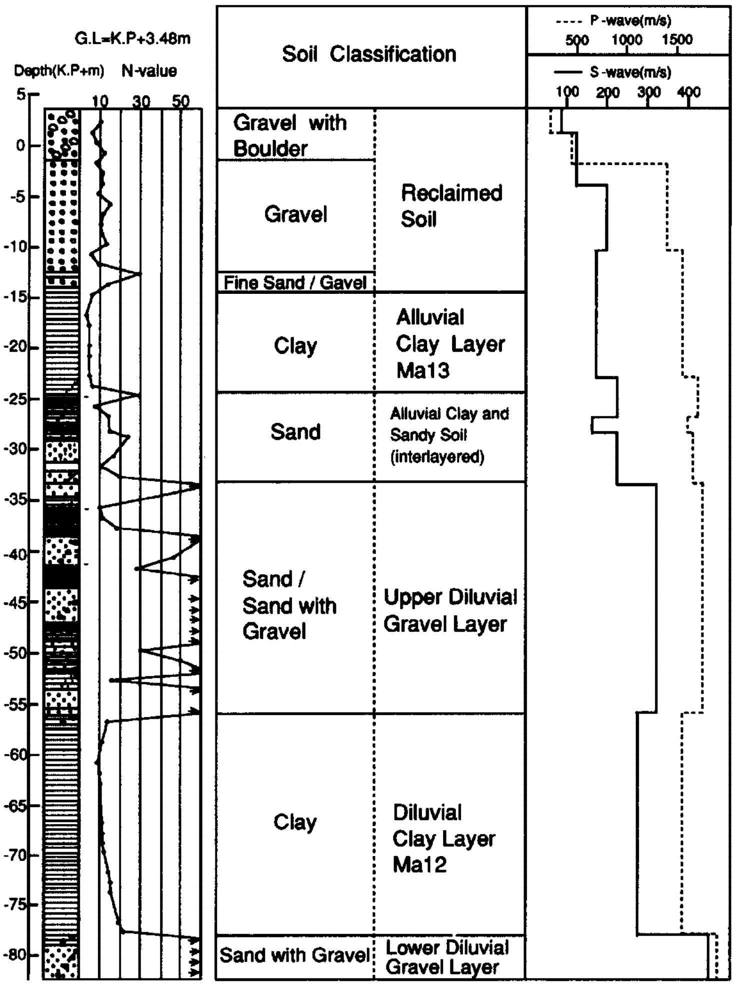

Figure 2: Soil profile in Port Island

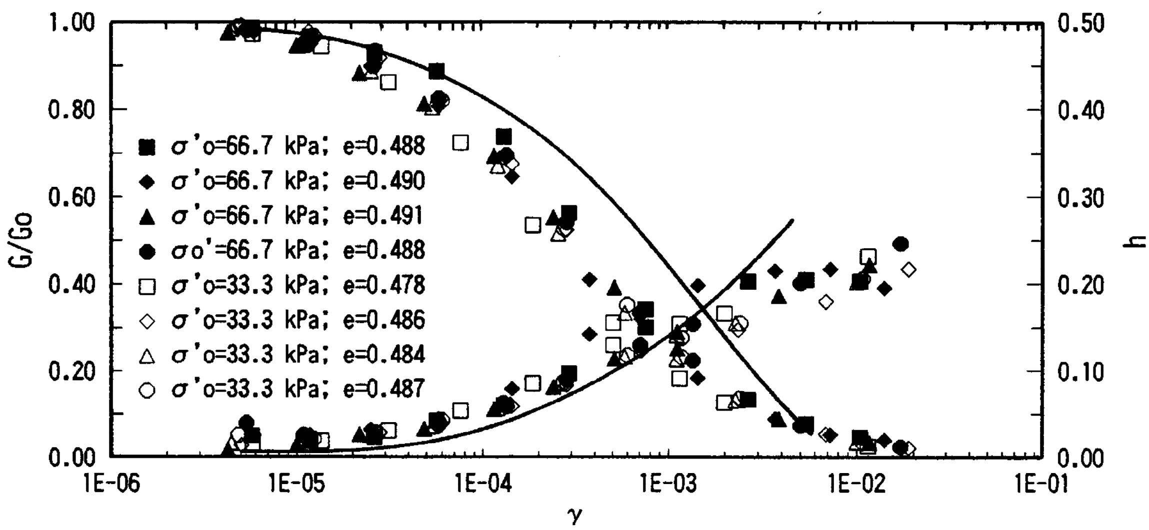

Figure 3: G/Go~h~ γ curves of weathered granite

frequency of cyclic loading was 0.1 Hz. Shear moduli and damping ratios at the tenth cyclic loading were taken as representative values in the present study.

Figure 3 shows the relation between the shear modulus and damping ratio with the shear strain. The shear modulus is normalized with respect to Go, i.e., shear modulus at small shear strain level. Solid dots in the figure correspond to σ'_0= 66.7 kPa, while open dots are for σ'_0= 33.3 kPa. It can be seen that all data points, whether for shear modulus or for damping, lie more or less along a single curve, indicating that the magnitude of confining pressure has negligible effect. Also shown in the figure by solid lines are the strain-dependent shear modulus and damping characteristics of the Japanese standard Toyoura sand for σ'_0= 66.7 kPa (from Kokusho, 1980). Note that the variation of damping with strain for the weathered granite deposit is almost similar to that of Toyoura sand. The degradation curves are also more or less similar except that the degradation for the weathered granite is more prominent specially in the range of strain from 10^{4} ~10^{-3}.

CYCLIC LOADING TEST RESULTS

In addition, a series of cyclic loading tests were performed using the torsional shear test apparatus. The dimensions of the specimens and the method of sample preparation were similar to those in the dynamic tests mentioned above. In order to examine the effect of load irregularity on the cyclic behavior of weathered granite, two types of loading patterns were employed: uniform harmonic loading and irregular loading using the same time history recorded on the ground surface at Port Island during the earthquake. For both loading cases, two sets of specimens were prepared: loose samples (e=0.483~0.491) and medium dense samples (e=0.347~0.354). In the tests, the specimens were isotropically consolidated to an effective confining stress of 100 kPa. Following consolidation, the specimens were subjected to a constant amplitude load with frequency of 0.1 Hz or an irregular time history of shear stress under undrained condition. The results of the tests for each loading type are discussed below.

Uniform Loading

Typical records of uniform loading tests obtained on loose sample (e=0.483) and medium dense sample (e=0.354) are shown in Figures 4(a) and 4(b), respectively. It can be seen in Figure 4(a) that for loose sample, the pore water pressure developed to become almost equal to the initial confining stress, with large deformation taking place in the sample. However, for medium dense sample, such phenomenon did not occur and shear deformation increased gradually as the cyclic stress application continued, even after the pore water pressure had become equal to the initial confining pressure. All the results of the uniform loading tests are summarized in Figure 5 in terms of the cyclic stress ratio, τ_d/σ'_0, plotted versus the number of cycles required to produce a double amplitude of shear strain of 7.5%. In the figure, τ_d corresponds to the deviator stress while σ'_0 is the initial confining pressure. It is noted that the smaller the void ratio (or the larger the relative density), the

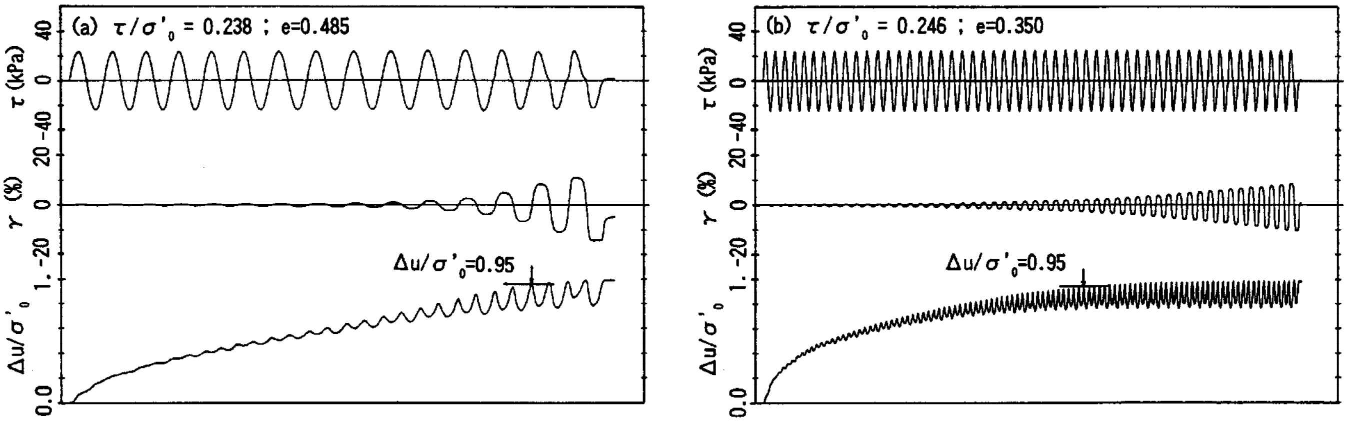

Figure 4: Typical results of uniform loading test on (a) loose sample; (b) medium dense sample

greater is the cyclic stress ratio required to induce the specified double amplitude shear strain for any number of loading cycle.

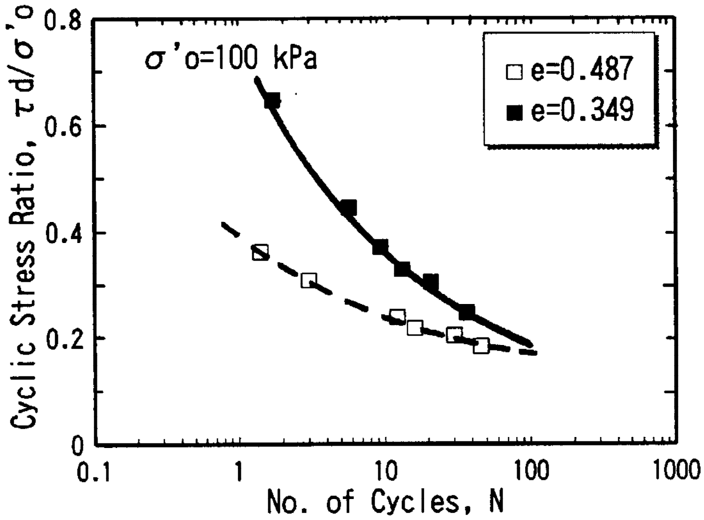

Figure 5: Cyclic strength curves for D.A. γ =7.5%

Irregular Loading

As shown in previous studies (e.g., Ishihara and Yasuda, 1975; Nagase and Ishihara, 1987), any time change in the shear stress in the soil deposit at shallow depths take place instantaneously in unison with the time variation of acceleration on the ground surface. At any instant of time, the only difference is the relative magnitude between the acceleration and the shear stress. Therefore, it follows that the time histories of shear stress having the same pattern as those of recorded accelerations can be applied to the soil sample in the torsional shear test to represent the in-situ stress changes during earthquakes.

In the present test, the recorded accelerations on the ground surface in Port Island are employed. The trajectory traced on the horizontal plane by the combination of the NS and EW components of the acceleration time histories are shown in Figure 6. It can be seen from the figure that the predominant shaking is along the NW-SE direction. The acceleration component projected in this direction is depicted in Figure 7. This acceleration time history is considered similar to the irregular pattern of shear stress changes which are applied to the specimen.

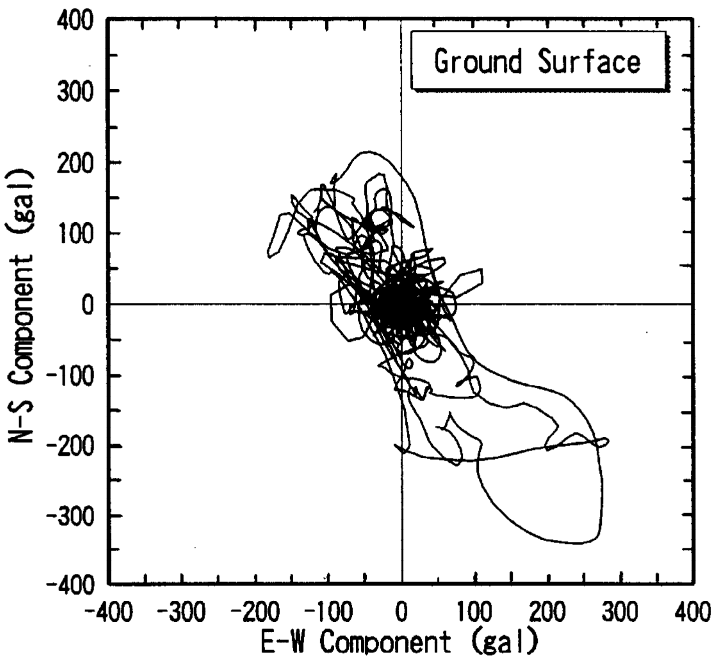

Figure 6: Trajectory of recorded surface accelerations in Port Island

A typical test result on loose sand (e=0.486) employing the irregular time history of acceleration is demonstrated in Figure 8(a). The amplitude of maximum irregular shear stress applied was τ_d/σ'_0=0.385, and the

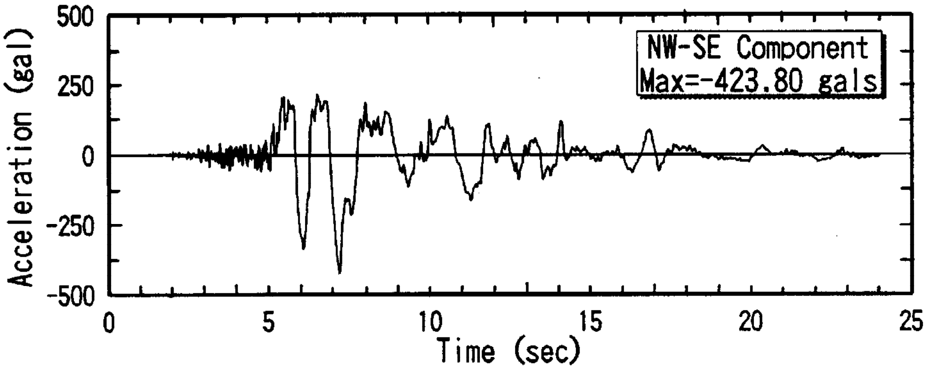

Figure 7: Acceleration time history in NW-SE direction

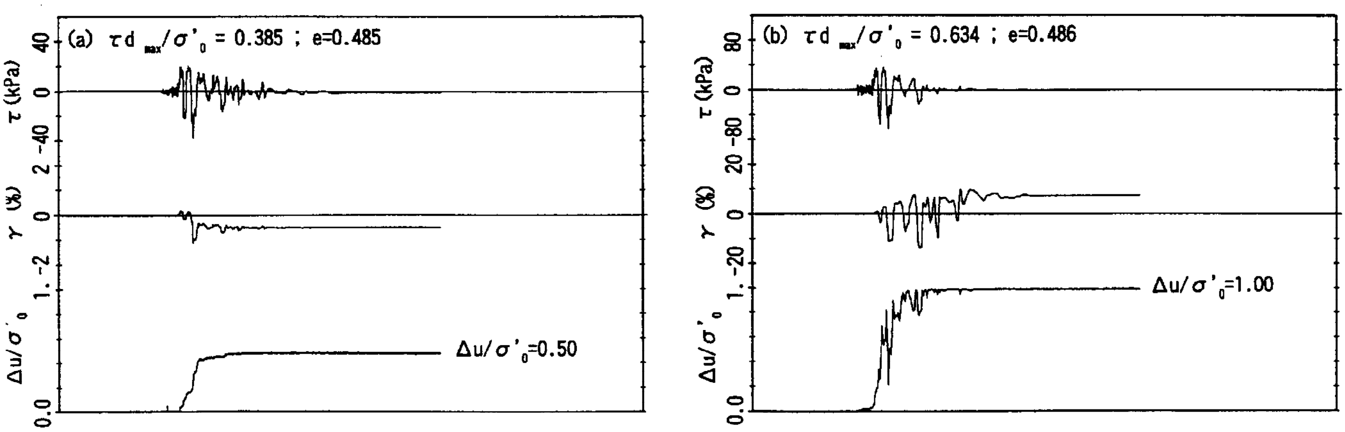

Figure 8: Typical results of irregular loading test on loose sample: (a) τ_d/σ'_0=0.385; (b) τ_d/σ'_0=0.634

pore water pressure and shear strain, γ, were recorded. It can be seen in the figure that when the peak shear stress is encountered, the pore water pressure builds up to about 50% of the initial confining stress, but it ceases to increase thereafter although shear stresses with lesser amplitude are still being applied to the specimen. In addition, the maximum shear strain attained is about 1.1% which occurred at the instant of maximum shear stress. Figure 8(b), on the other hand, depicts the results for a specimen with identical void ratio but subjected to higher stress level, with the maximum shear stress ratio of τ_d/σ'_0=0.634. It can be seen that in this case, the pore water pressure builds up to a value almost equal to the confining pressure at the time when the peak shear stress is applied, indicating the attainment of initial liquefaction. In addition, a shear strain of about 10% occurs at the time of peak shear stress application, but it grows to a magnitude of 12% in the course of shear stress application following the peak.

<u>Shear Strains Induced by Irregular Load</u>

The magnitude of the maximum shear strain developed in the specimen can be considered as a key parameter for representing the strength characteristics of deposits under cyclic loading conditions (Nagase and Ishihara, 1987). The plots of the maximum shear strain with the magnitude of irregular shear stress expressed in terms of the maximum shear stress ratio, τ_{max}/σ'_v, for both loose and medium dense samples are shown in Figure 9 by open and solid circles, respectively. It may be seen that for both cases, the maximum shear stress ratio increases abruptly and then gradually with the shear strain. In order to produce similar magnitude of maximum shear strain, the denser sample requires a greater level of shear stress than a looser sample. The figure also illustrates the test results for uniform loading test with 20 cycles of load application (shown by open and solid squares). By comparing the curves for irregular and uniform loadings, the effect of the irregular nature of seismic load application on the strength of soil may be evaluated. This can be done by comparing the maximum shear stress ratio required to cause a given amount of maximum shear strain under irregular loading conditions with the cyclic stress ratio required to produce the same amount of shear strain with 20 cycles of load

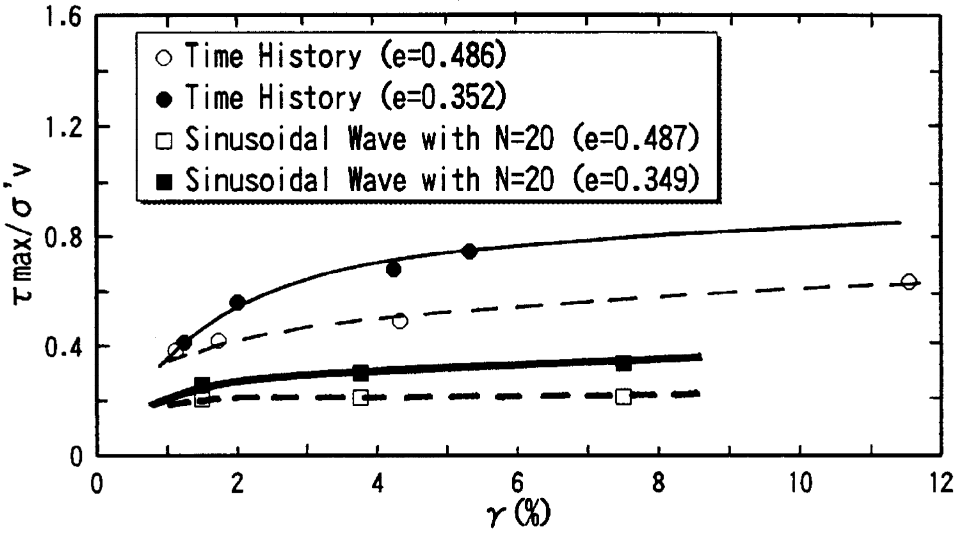

Figure 9: Variation of maximum shear stress ratio, τ_{max}/σ'_v, with shear strain

application under uniform loading conditions. The ratio between these two stress levels is referred to as the load irregularity factor, τ_d/τ_{max}. From the figure, it can be seen that the load irregularity factor changes with the amplitude of shear strain being considered but lies by and large between 0.43~0.50 for loose samples (e=0.486) and between 0.40~0.44 for medium dense samples (e=0.350). For a value of γ_{max}=3.75% (single amplitude), the load irregularity factor is equal to 0.46 for e=0.486 and 0.42 for e=0.350. These factors correspond to values of C_2 (reciprocal of load irregularity factor) in the Japanese code for bridge design of 2.2~2.4, which are generally larger than the recommended values of 1.4~1.8. Clearly, the results show that the present criteria in evaluating liquefaction strength of weathered granite on the basis of cyclic stress ratio causing initial liquefaction in 20 cycles of uniform load application may be inadequate.

Volume Change During Reconsolidation

The volume change characteristics of the sample during dissipation of pore water pressures following the application of cyclic shear stress in undrained condition is also investigated. In the test series mentioned above, after the irregular load with a certain amplitude was applied to the sample and the pore water pressure and the maximum shear strain produced have been measured, the drainage line from the test sample was opened to dissipate the residual pore water pressure that have developed and the volume change of the sample due to reconsolidation was monitored. Plots showing the variation of ε_v with the maximum shear strain, γ, is shown

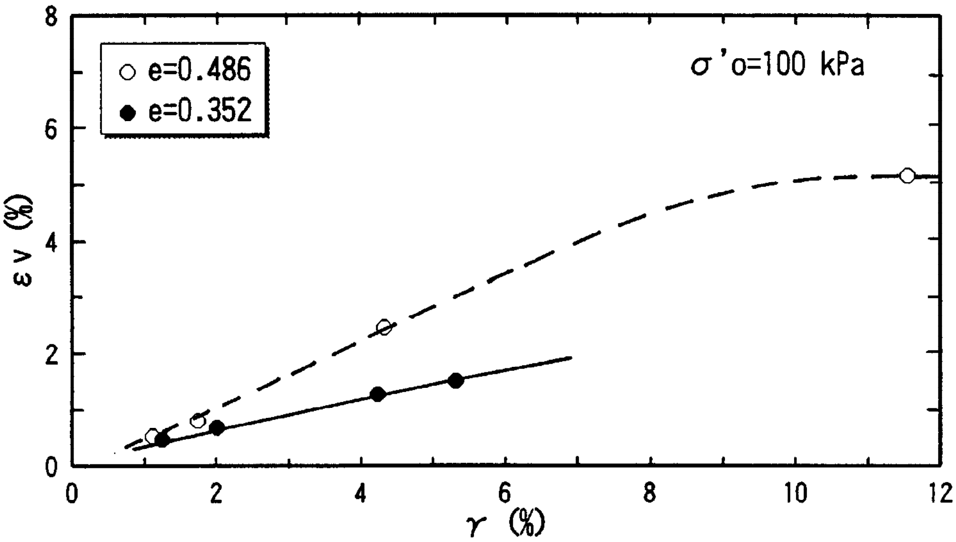

Figure 10: Variation of volumetric strain, ε_v, with shear strain

in Figure 10 for loose and medium dense samples. It can be observed from the figure that for a particular level of shear strain, the volumetric strain is larger for loose samples than for medium dense samples. Comparison of these results with those compiled by Ishihara and Yoshimine (1992) yielded an approximately coincident trend in correlating the reconsolidation volume change with the maximum shear strain during the undrained cyclic loading.

CONCLUDING REMARKS

In order to clarify the dynamic behavior of weathered granite soil deposit which suffered extensive damage during the 1995 Kobe Earthquake, a series of torsional shear tests were performed. Dynamic tests were conducted to determine the shear strain-dependent properties of the deposit. The results showed that the variation of shear modulus and damping with shear strain for weathered granite is comparable to those of the Japanese standard sand. In addition, the cyclic tests conducted using uniform and irregular time histories resulted in values of the irregularity factor C_2 which are much greater than those recommended in the current Japanese code. Further tests may be necessary to support the above findings.

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REFERENCES

Ishihara, K. and Yasuda, S. (1975). Sand Liquefaction in Hollow Cylindrical Torsion under Irregular Excitation. *Soils and Foundations* Vol. 15, No. 1, 45-59.

Ishihara, K. and Yoshimine, Y. (1992). Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes. *Soils and Foundations* Vol. 32, No. 1, 173-188.

Kokusho, T. (1980). Cyclic Triaxial Test of dynamic Soil Properties for Wide Strain Range *Soils and Foundation* Vol. 20, No. 2, 45-60.

Nagase, H. and Ishihara, K. (1987). Effects of Load Irregularity on the Cyclic Behavior of Sand. *Soil Dynamics and Earthquake Engineering* Vol. 6, No. 4, 239-249.