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UNDRAINED FLOW CHARACTERISTICS OF PARTIALLY SATURATED SANDY SOILS IN TRIAXIAL TESTS

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ABSTRACT

It became known that the partially saturated soil layers of about 5 metres in thickness prevail beneath the groundwater levels. This is evident from the data of velocity logging tests, where the propagation velocity of primary waves, V_p, takes a value of 500 to 1000 m/s at such soil layers, which is well below the value of 1500 m/s observed under fully saturated conditions and implies that such soil layers are in partially saturated conditions. The liquefaction resistance of partially saturated sands has been examined extensively in laboratory tests, and attempts have been made to incorporate the effects of partial saturation on the liquefaction potential of sandy soil layers located immediately beneath the groundwater levels. In the present study, the flow characteristics of partially saturated sands are examined. One case study is provided, in which the flow potential at a particular site is evaluated. Soil samples were retrieved from a site, and laboratory triaxial tests were conducted. The results of laboratory tests are described with respect to the evaluation of the flow potential at this site.

Keywords: Flow potential, sand, partial saturation, triaxial test

INTRODUCTION

The primary interests of geotechnical engineering practice regarding the stability of earth structures during earthquakes are twofold. The first one is obviously the liquefaction potential of a particular soil layer, concerning the possibility of occurrence of soil liquefaction. The second one is what is called flow potential, which is concerned about whether flow-type failures would occur subsequently during earthquakes.

It is known that the partially saturated soil layers of about 5 metres in thickness prevail beneath the groundwater tables. This is evident from the data of velocity logging tests, where the propagation velocity of primary waves, V_p, takes a value of 500 to 1000 m/s at such soil layers, which is well below the value of 1500 m/s observed under fully saturated conditions and implies that such soil layers are in partially saturated conditions. The liquefaction resistance of partially saturated sands has been examined extensively in laboratory tests, (Tsukamoto et al. 2002 and others). In addition, attempts have been made to provide several field case studies, which were aimed at incorporating the effects of partial saturation on the evaluation of liquefaction potential of sandy soil layers located immediately beneath the groundwater tables, (Nakazawa, et al. 2004 and others). However, since it is usually at such partially saturated soil layers located immediately below the groundwater tables that flow failures would be induced during earthquakes, the influence of partial saturation needs to be also incorporated in the evaluation of flow potential at such soil layers. The flow deformation of partially

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saturated Toyoura sand was examined based on the results of laboratory triaxial tests, and the characteristics of the residual strength were clarified in the past study, (Kamata et al. 2006). Therefore, it would be worthwhile to carry out some field case studies on the possibility of flow occurrence taking into account the influence of partial saturation.

The present study is aimed at providing some field case study, in which the residual shear strength of partially saturated soils is examined from the results of laboratory triaxial tests and the possibility of flow occurrence is evaluated incorporating the effects of partial saturation.

FIELD CONDITIONS AND SOIL SAMPLING

Figure 1 shows the location where the undisturbed deep soil sampling was carried out. This site is located along Tokyo Bay. The soil profile with depth at this site is indicated in Fig.2. The groundwater table is found at a depth of about 10.5 metres below the ground surface. It is not certain whether it would be worthwhile to examine the possibility of flow occurrence at such a site where the ground water table is located deep. However, since high-quality soil sampling was carried out at this site, a series of triaxial tests were conducted in the present study and the test results are discussed in detail below with respect to the flow deformation and residual shear strength. Located immediately below the groundwater table is the layer of fine sand, at which the P-wave velocity changes from 750 m/s to over 1600 m/s. Soil sampling was intended from a depth of 14 metres where the P-wave velocity of 750 m/s was observed. Performed here with a help of the geotechnical consulting firm was what is called "gel-push" sampling, which has been invented as one of the undisturbed soil sampling methods without freezing process of soil samples.

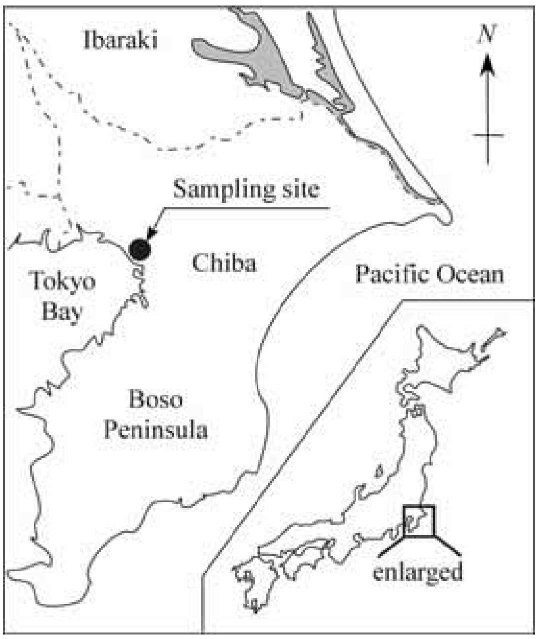


Figure 1. Location of site of sampling

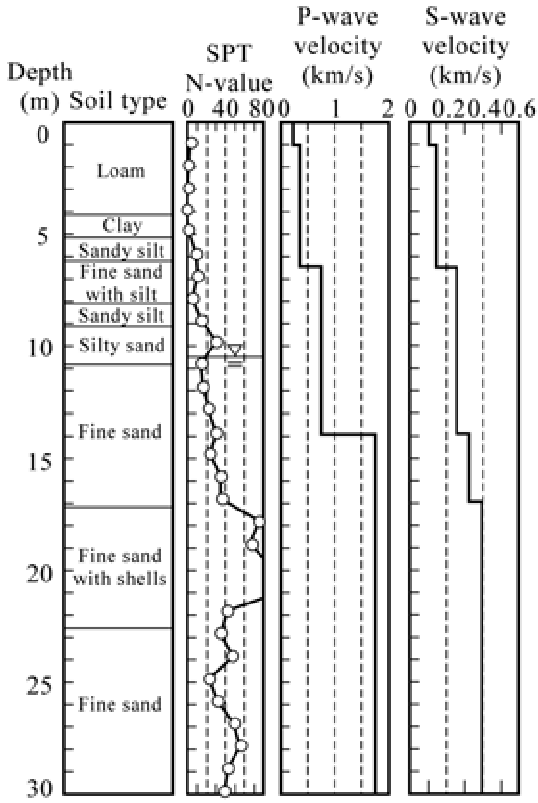


Figure 2. Soil profile with depth at the sampling site

DETAILS OF LABORATORY TESTS

Soil materials used

The grain size distributions of the soils are shown in Fig. 3. The physical properties of the soil retrieved from the sampling site are as follows, specific gravity G_s = 2.712, fines content less than 0.075 mm F_c = 14%, mean particle diameter D_{50} = 0.15 mm, maximum and minimum void ratios e_{max} = 1.464 and e_{min} = 0.827. For comparison purposes, the results of laboratory triaxial tests on Toyoura sand are also shown below. Toyoura sand is a clean fine sand with no fines, and the physical properties are as follows, specific gravity G_s = 2.641, mean particle diameter D_{50} = 0.16 mm, maximum and minimum void ratios e_{max} = 0.971 and e_{min} = 0.607.

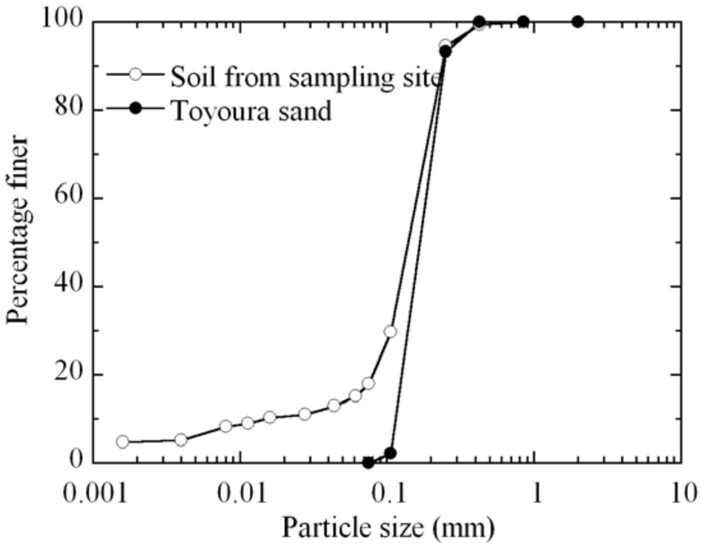


Figure 3. Grain size distribution of soils

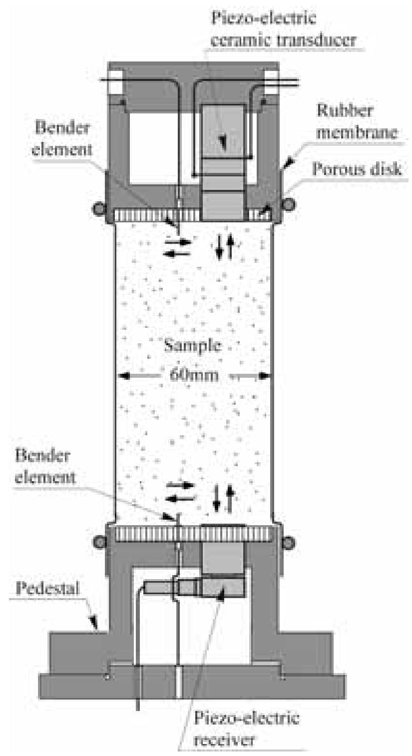
Degree of partial saturation

The coefficient of pore water pressure, B-value, is frequently used to ensure the full saturation of soil specimens in laboratory triaxial tests. The B-value measures a response of excess pore pressure to a designed increase of the effective overburden stress, and is found to be a good parameter indicative of the degree of partial saturation corresponding to the saturation ratio of about S_r = 90% to 100%. However, from the practical viewpoint, since the B-value cannot be measured directly in the fields, it was proposed that other parameters such as the propagation velocities of longitudinal and shear waves, V_p and V_s, be alternatively used in estimating the degree of partial saturation in the fields from the following expression, (Tsukamoto et al. 2002, Nakazawa et al. 2004),

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

where ν_b is the skeleton Poisson’s ratio. The above equation implies that the B-value can be inferred from the propagation velocities, which are obtained from field velocity logging tests, by assuming a typical value of ν_b.

In the present study, the measurements of velocities of longitudinal and shear waves were conducted in the triaxial tests using the equipment shown in Fig. 4. This testing equipment effectively consists of a piezo-electric transducer and an accelerometer transmitting and receiving compressional waves, and a pair of bender elements transmitting and receiving shear waves. The values of V_p and V_s measured on the undisturbed samples are shown in Fig. 5. The data on reconstituted samples are also included. Based on the comparison between the data and the theoretical relations of Eq. (1), the skeleton Poisson’s ratio ν_b that can be used for this soil seems to take a value of 0.45.



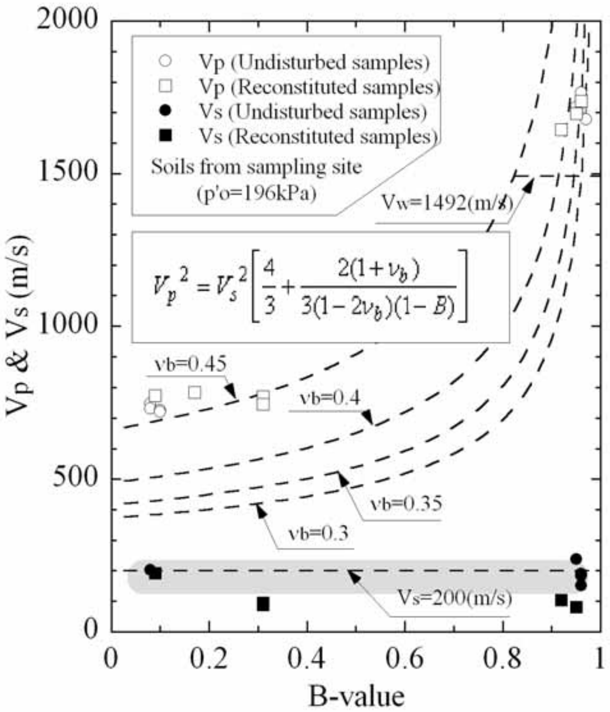


Figure 4. Testing equipment measuring V_p and V_s, (Tsukamoto et al. 2002)

Figure 5. Plots of V_p and V_s against B-value for the soil samples retrieved

Testing procedure

As a result of the undisturbed soil sampling as described above, four cylindrical blocks of soils with 19 cm in diameter and 12 cm in height were retrieved from the depth of 14 metres and provided for laboratory tests. Each block of soils was first adequately frozen, and the frozen soil blocks were trimmed into four pieces of triaxial specimens with 6 cm in diameter and 12 cm in height. In total, 16 undisturbed triaxial specimens have become available for laboratory tests. Multiple series of triaxial tests on fully saturated specimens as well as partially saturated specimens were planned, where isotropically consolidated p-constant undrained compression and extension tests were performed to examine the flow potential and undrained cyclic tests were conducted to examine the liquefaction potential. The benefits of conducting p-constant tests on partly saturated soils were described by Tsukamoto et al. (2002). In the present study, the results of the test series of undrained compression and extension tests are described and the flow characteristics of the soil are examined.

The frozen soil specimens have been placed in the triaxial cell chamber for at least three hours, and isotropically consolidated to a confining stress of σ'_o = 196 kPa, considering the effective overburden stress expected at the depth of soil sampling. The degree of saturation was then controlled by monitoring the value of V_p, and the corresponding B-value was measured. In one of the test series, fully saturated specimens were prepared with the value of V_p greater than 1500 m/s and the B-value greater than 0.95, and the undrained triaxial compression and extension tests were conducted. In the other test series, partly saturated specimens were prepared with the value of V_p equal to around 750 m/s and the corresponding B-value equal to 0.1 to 0.15, which is equivalent to the value observed at the depth of soil sampling in the field velocity logging test. The undrained triaxial compression and extension tests were then carried out. The test series described in the present study are listed in Table 1.

After the tests on the undisturbed specimens, the void ratio measurements were done. The undisturbed specimens are found to have the relative densities of D_r = 77 to 84 % at the time of consolidation.

Table 1. Test series on undisturbed samples

Initial degree of saturation	B-value	V_p (m/s)	Shearing mode
Fully saturated	>0.95	>1500	Triaxial compression (TC)
			Triaxial extension (TE)
Partly saturated	0.1 – 0.15	750	Triaxial compression (TC)
			Triaxial extension (TE)

LABORATORY TEST RESULTS

Undrained compression and extension tests

From the results of undrained triaxial tests, it has been customary to characterize the conditions of flow deformation by distinguishing the stress – strain behaviours into “flow”, “flow with limited deformation” and “no flow”, as shown in Fig. 6. The same principle is adopted herein in the interpretations of the results of undrained triaxial tests shown below.

The results of the undrained triaxial compression tests are shown in Fig. 7. In Fig. 7(a), the responses of the excess pore water pressure, u_w, are plotted against the axial strain, ε_a. It is seen that the positive pore water pressure first develops to achieve a peak and then begins to reduce. It is at this peak point that the state of phase transformation is defined. The excess pore water pressure then turns into negative. The initial stage of tests is defined as “u>0” zone, where the positive pore pressure is observed, while the rest of tests is defined as “u<0” zone, where the negative pore pressure is observed. When the responses of the fully saturated specimen and the partly saturated soil specimen are compared, it appears that the excessive development of the pore water pressure is prevented in the partly saturated condition, regardless of whether the pore water pressure is positive or negative.

In Fig. 7(b), the plots of deviatoric stress, $q = \sigma_1 - \sigma_3$, against the axial strain are shown. It is seen that the partly saturated specimen showed a weaker response than the fully saturated specimen, due primarily to the fact that the negative pore pressure developed in the partly saturated specimen is lower than that in the fully saturated specimen. Based on the characterization shown in Fig. 6, the stress – strain relations can be categorized as “no flow”, regardless of whether the soil specimens are fully saturated or partly saturated.

The effective stress paths are shown in Fig. 7(c), where $p' = (\sigma'_1 + 2\sigma'_3)/3$. It appears that the failure envelopes for the fully saturated specimen and partly saturated specimen are the same. The value of M_{ss} is therefore uniquely determined, so is the value of M_{pt}. The residual shear strength ratio, τ_m/p'_o, is conventionally defined at the state of phase transformation, and is expressed as follows,

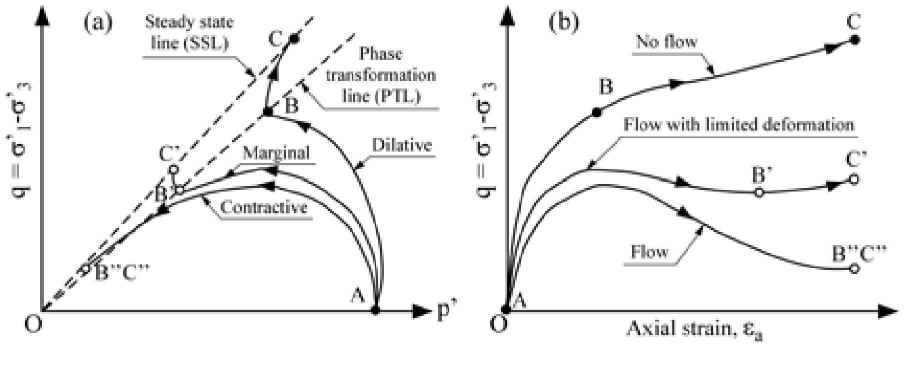
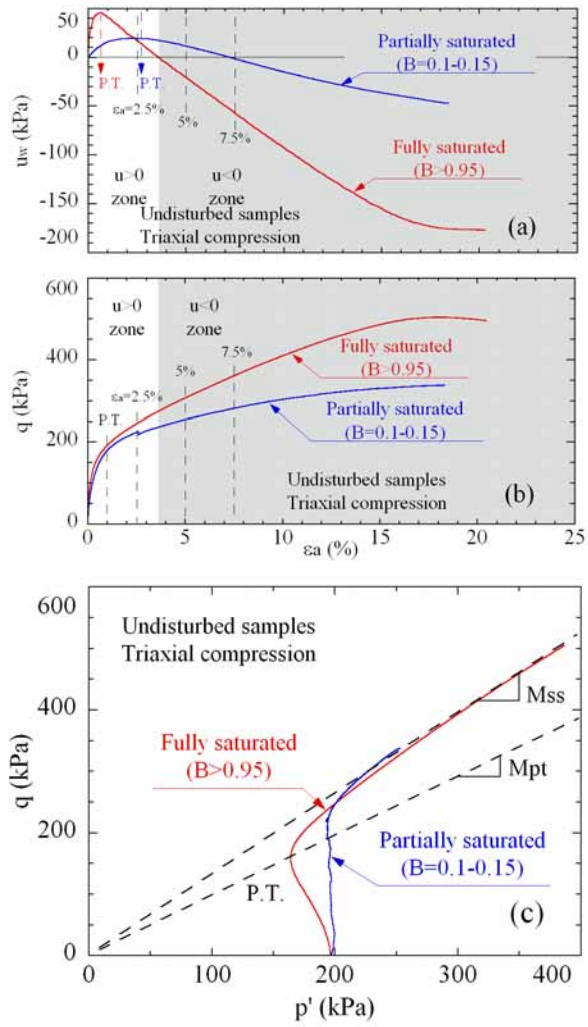


Figure 6. Characterization of flow deformation in triaxial tests



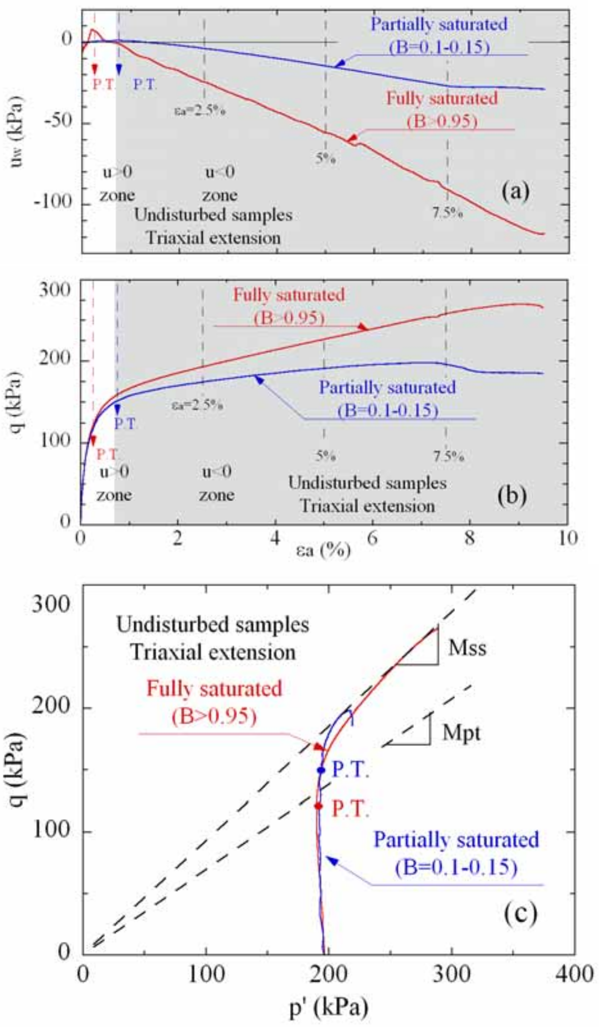


Figure 7. Results of undrained triaxial compression tests, (Undisturbed samples)

Figure 8. Results of undrained triaxial extension tests, (Undisturbed samples)

$$\frac{\tau_m}{p'_o} = \frac{q_{pt}}{2\,p'_o} = \frac{q_{pt}}{2\,p'_{pt}}\frac{1}{r_c} = \frac{M_{pt}}{2\,r_c} \;, \quad (2)$$

where p'_o is the initial effective mean principal stress, p'_{pt} and q_{pt} are the effective mean principal stress and deviatoric stress at states of phase transformation, r_c is the initial state ratio defined as $r_c = p'_o/p'_{pt}$, and M_{pt} is the inclination of the failure envelope, $M_{pt} = q_{pt}/p'_{pt}$. Since the value of M_{pt} is found to be uniquely determined, it is the value of r_c and hence the difference in the response of the pore pressure that leads to the difference in the residual shear strength ratio.

The same set of the diagrams is shown in Fig. 8 for undrained triaxial extension tests. The same observations are basically made. In the effective stress paths shown in Fig. 8(c), there appear to be no distinctive points where the states of phase transformation (P.T.) can be defined. However, the P.T. states are determined based on the levels of axial strain where the maximum positive pore pressures are observed in the plots of $u_w - \varepsilon_a$ shown in Fig. 8(a). From the results shown in Fig. 8(b), the stress – strain relations can be categorized as “no flow”, regardless of whether the soil specimens are fully saturated or partly saturated.

From the results of q – ε_a shown in Figs. 7(b) and 8(b), the values of deviatoric stress, q, at various levels of axial strain including the states of P.T., u_w=0, ε_a=2.5%, 5% and 7.5% are read off, and the values of the residual shear strength ratio, τ_m/p'_o, defined in Eq. (1) are calculated. The values of τ_m/p'_o thus obtained are plotted against the B-value, as shown in Figs. 9 and 10. From the general observations in the previous study on Toyoura sand, in the "u>0" zone, the residual shear strength ratio tends to increase as the B-value reduces, while it tends to decrease as the B-value reduces in the "u<0" zone, (Kamata et al. 2006). However, there seems to be no such definitive tendency in the results shown in Figs. 9 and 10 for the undisturbed soil samples.

In Fig. 11, the values of the residual shear strength ratio, τ_m/p'_o, at states of phase transformation are plotted against the B-value. The white round points indicate the data for triaxial compression (TC), and the dark round points for triaxial extension (TE). The reference lines are also provided, which extrapolate the results on Toyoura sand with various relative densities and shearing modes of TC or TE. These reference lines are based on the data of the previous study, (Kamata et al. 2006). Based on the comparison with the reference lines, the residual shear strength ratio of the undisturbed samples is almost equivalent to that of Toyoura sand with the relative density D_r of over 60%.

It was found in the previous study on Toyoura sand that the threshold values of the residual shear strength ratio, τ_m/p'_o, giving the boundaries between contractive behaviour, marginal behaviour and dilative behaviour are independent of the B-value and uniquely determined, (Kamata et al. 2006). Herein, the contractive behaviour corresponds to "flow", while the dilative behaviour corresponds to "no flow". The reference lines indicating such boundaries are included in Fig. 12, in which the same set of data as shown in Fig. 11 are also plotted. The results of the undisturbed samples are located in the zone of dilative behaviour, and therefore the occurrence of flow deformation appears to be not likely.

Based on the reference lines shown in Fig. 11, the general tendency is that the residual shear strength ratio, τ_m/p'_o, defined at P.T. states tends to gradually increase with the reduction in the B-value. In order to better evaluate such tendency, the ratios of the residual shear strength ratio at P.T. states for partially saturated conditions to that for fully saturated conditions are calculated and plotted against the B-value, as shown in Fig. 13. The reference lines for triaxial compression, triaxial extension and the average of these two on Toyoura sand are also included in this diagram. It appears that the undisturbed samples do not show significant strength increase as compared with the reference lines. Therefore, more case studies would be necessary to establish such strength increase with reducing B-value.

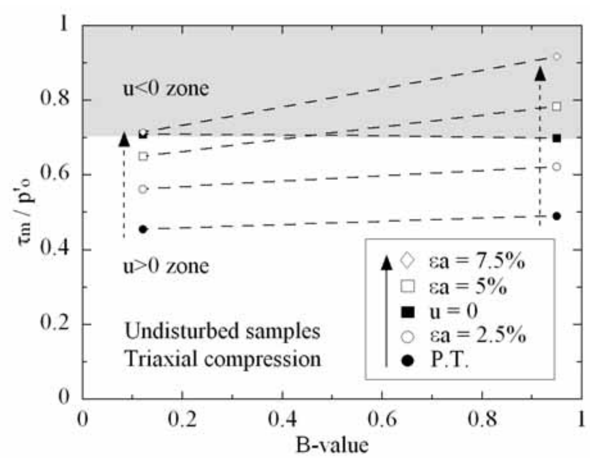


Figure 9. Plots of undrained shear strength ratio at various levels of axial strain ε_a against B-value, (Triaxial compression)

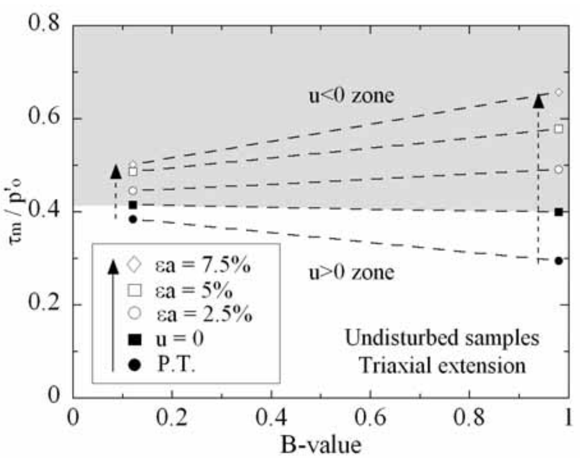


Figure 10. Plots of undrained shear strength ratio at various levels of axial strain ε_a against B-value, (Triaxial extension)

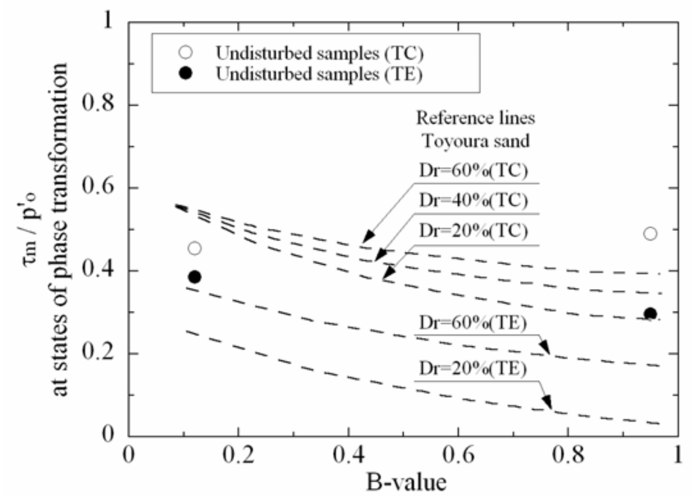


Figure 11. Plots of undrained shear strength ratio at states of P.T. against B-value, (Comparison with Toyoura sand)

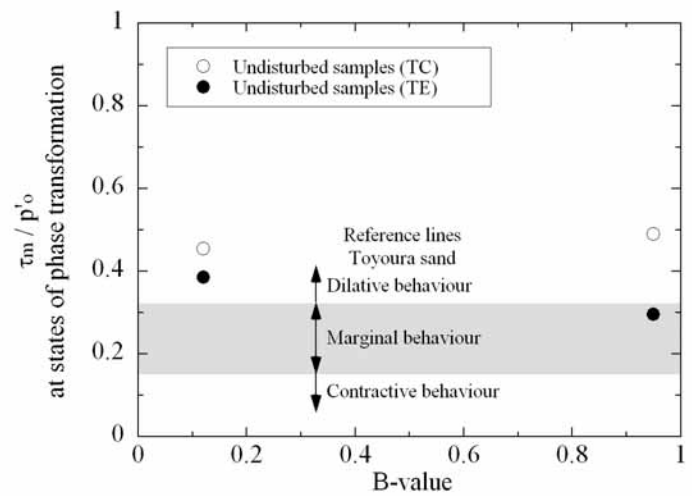


Figure 12. Plots of undrained shear strength ratio at states of P.T. against B-value, (Contractive and dilative behaviours)

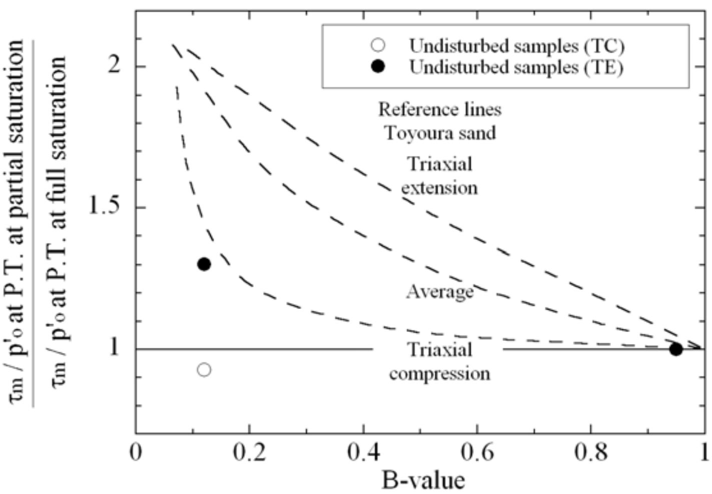


Figure 13. Plots of ratio of undrained shear strength ratio at states of P.T. for partly saturated condition to that for fully saturated condition against B-value

CONCLUSIONS

In order to evaluate the flow occurrence at a particular soil deposit during earthquakes, one case study was provided in the present study. The present study was particularly focused on examining the effects of partial saturation on the condition of flow. Four blocks of undisturbed soil samples were retrieved from the soil deposits located in Tokyo Bay area by means of what is called gel-push" sampling, and a dozen of undisturbed triaxial specimens were produced. Based on the results of undrained triaxial compression and extension tests on fully saturated specimens as well as partly saturated specimens, the flow occurrence was examined with respect to the types of stress – strain behaviour characterized as contractive, marginal and dilative bahaviours. The level of the residual shear strength ratio leading to flow occurrence was discussed in detail. Herein, the discussion was given with respect to the results of reconstituted Toyoura sand, and therefore further laboratory study would be desirable on reconstituted soil samples retrieved from this site.

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