EVALUATION OF UNDRAINED SHEAR STRENGTH OF SOILS FROM FIELD PENETRATION TESTS

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ABSTRACT

The evaluation of undrained shear strength of soils is necessary in determining the possibility of occurrence of flow deformation during earthquakes. The present study is aimed at examining the evaluation of undrained shear strength of silty sands from field with Swedish weight sounding tests and cone penetration tests. Based on the outcome of the previous studies on laboratory triaxial tests, the undrained shear strength ratio is defined as the undrained shear strength divided by the initial effective major principal stress. The undrained shear strength ratio is then formulated with respect to the relative density. The penetration resistances of Swedish weight sounding and cone penetration tests are then formulated with respect to the effective overburden stress and relative density, based on laboratory calibration chamber tests. By combining these formulations, the correlations of the undrained shear strength with Swedish penetration resistance and cone tip resistance are established. The range of values of penetration resistances indicative of soil layers susceptible to flow deformation is discussed. The correlations of the undrained shear strength with field penetration resistances thus derived are then examined from case history studies. Two case history studies are carried out with Swedish weight sounding tests at the sites of flow failures induced during the recent earthquakes. A series of case history studies are reexamined, which were carried out with Dutch cone penetration tests in the past studies.

Key words: sandy soil, sounding, triaxial test, undrained shear strength (IGC: C3/D6)

INTRODUCTION

The procedures for estimating liquefaction resistance of soils, which are necessary for evaluating the possibility of occurrence of soil liquefaction during earthquakes, have been examined extensively, based on laboratory cyclic triaxial tests on frozen intact samples and also on field penetration tests including standard penetration tests (SPT), cone penetration tests (CPT) as well as velocity logging tests. To evaluate the occurrence of flow deformation during earthquakes, the procedures for estimating undrained shear strength of soils are necessary and have also been examined, based on laboratory triaxial tests and also on case history studies involving field penetration tests, (Seed, 1987; Seed and Harder, 1990; Ishihara et al., 1990a; Idriss and Boulanger, 2007; and others). One of the recent findings was that, in the presence of a soil layer with significantly lower permeability overlying the liquefiable layer, there would be situations where the upward pore water seepage during earthquakes could lead to localized loosening of the liquefiable soil, resulting in the undrained shear strength mobilized in the field being much lower than that observed in laboratory tests, (Idriss and Boulanger, 2007). Such a phenomenon was termed as ‘water film generation’ and

‘void redistribution mechanisms’, (Kokusho, 2000; Kulasingam et al., 2004; Malvick et al., 2006; and others).

The characterization of soil properties using field penetration tests has recently evolved considerably. The use of calibration chamber tests allowed the effects of state parameters such as soil density and overburden stress as well as grain composition of soils on the field penetration resistance to be examined in detail. On the other hand, the effects of soil density, confining stress and grain composition of soils on the undrained shear strength of saturated sands have also been examined extensively by using laboratory triaxial tests. Despite the difficulty in estimating the undrained shear strength mobilized in the field due to the complex phenomena of void redistribution mechanisms, as mentioned above, it would be worthwhile to combine such independent studies conducted by laboratory calibration chamber tests and triaxial tests, which would allow the undrained shear strength to be directly correlated with the field penetration resistance of soils.

The present study is aimed at examining the direct correlations of the undrained shear strength of silty sands with the field penetration resistances of Swedish weight sounding tests and cone penetration tests.

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Fig. 1. Typical behaviour in undrained triaxial compression and its characterization, (a) effective stress paths and (b) stress-strain relations

UNDRAINED SHEAR STRENGTH

In what follows, the undrained shear strength of silty sands is examined based on the previous studies composed of a multiple series of undrained triaxial tests on anisotropically consolidated saturated specimens of clean fine sand and various silty sands, (Tsukamoto et al., 2004a; Tsukamoto et al., 2008). The general framework and outcome of the previous studies are first described. The undrained shear strength of silty sands is then formulated with respect to the relative density, D_r , in a form suitable for the purpose of the present study.

Flow Condition and Undrained Shear Strength Ratio

In the previous studies, the flow conditions were identified by categorizing the undrained behaviour in terms of contractive and dilative behaviour, as follows. The typical behaviour of anisotropically consolidated sand subjected to undrained triaxial compression is schematically illustrated in Fig. 1. The effective stress paths are shown in Fig. 1(a) in the plots of effective mean stress, $p' = (\sigma_1' + 2\sigma_3')/3$, and deviatoric stress, $q = \sigma_1' - \sigma_3'$. The stress-strain relations are shown in Fig. 1(b) in the plots of axial strain, ϵ_a , and q . The point A indicates that the specimens are anisotropically consolidated. The specimens would then follow different effective stress paths, depending upon their density and fabric structures. The dense specimen shows dilative behaviour, where it follows along the points B and C, and the shear stress continues to increase. The points B and C define the state of phase transformation and steady state. This behaviour is categorized as “no flow”, and the undrained shear strength is defined as the shear stress mobilized at the state of phase transformation. On the other hand, the loose specimen shows contractive behaviour, where it follows along the points B'' and C'', and the shear stress experiences peak and then reduces. The points B'' and C'' define the quasi-steady state and steady state. This behaviour is categorized as “flow”, and the undrained shear strength is defined at the quasi-steady state. There is intermediate behaviour, where it follows along the points B' and C'. This

behaviour is categorized as “flow with limited deformation”, and the undrained shear strength is defined at the quasi-steady state B', which is lower than that mobilized at the steady state C'.

The undrained shear strength ratio was then defined by normalizing the undrained shear strength, $S_{us} = q_s \cos \phi_s/2$, with respect to the initial major effective principal stress, σ'_{1c} , at consolidation, as follows,

$$\frac{S_{us}}{\sigma'_{1c}} = \frac{q_s}{2} \cos \phi_s \frac{1}{\sigma'_{1c}}, \quad (1)$$

where q_s and ϕ_s are the deviatoric stress and internal friction angle at state of phase transformation.

Formulation

The physical properties of the soils examined in the present study are summarized in Table 1, (Tsukamoto et al., 2004a, 2008). The grain size distributions of the soils are also shown in Fig. 2(a). Toyoura sand is clean fine sand with no fines, and all the other soils are fine sands with some amount of non-plastic fines. From a number of tests conducted in the previous studies, the plots of values of S_{us}/σ'_{1c} against the relative density, D_r , were deduced and each plot was identified either as “flow” or as “no flow”. The plots for Toyoura sand and Omigawa sand are shown in Fig. 3. The primary findings of the previous studies were as follows.

- (1) The border determining the conditions of “flow” and “no flow” can be given by a unique value of the undrained shear strength ratio, S_{us}/σ'_{1c} , regardless of the degree of anisotropic consolidation as well as the triaxial compression/extension modes.
- (2) This threshold value of the undrained shear strength ratio takes a value ranging from 0.24 to 0.26 for clean sand and ordinary silty sands.

In what follows, based on the plots of values of S_{us}/σ'_{1c} against D_r for the four different soils used in the previous studies, the effects of the relative density on the undrained shear strength are formulated as follows,

Table 1. Physical properties and inferred parameters of soils

	Laboratory triaxial tests				Laboratory calibration chamber tests			Case history studies	
	Toyoura sand	Omigawa sand No. 1	Jamuna river sand	Shirasu	Omigawa sand No. 2	Inage sand	Narita sand	Tsukidate	Tanno
Specific gravity G_s	2.657	2.694	2.745	2.337	2.698	2.546	2.691	2.548	2.465
D_{50} (mm)	0.17	0.17	0.19	0.20	0.15	0.15	0.13	0.25	0.19
Atterberg limits	NP	NP	NP	NP	NP	NP	NP	NP	NP
Fines content F_c (%)	0	8.4	15.5	24.0	10.7	23.8	29.1	36.2	28.4
Maximum void ratio $e_{\max}$	0.973	1.282	1.202	1.442	1.495	1.021	1.76	1.789	1.872
Minimum void ratio $e_{\min}$	0.607	0.796	0.602	0.766	0.884	0.509	0.995	0.947	0.977
$e_{\max} - e_{\min}$	0.366	0.486	0.6	0.676	0.611	0.512	0.765	0.842	0.895
D_{ro} (%)	0%	25%	25%	25%	25%	25%	25%		
$A_{us} = (S_{us}/\sigma'_{1c})/(D_r - D_{ro})^2$ Average	3.3	2.1	1.73	1.54					
under TC	5.9	5.4	2.19	2.78					
under TE	2.1	1.1	1.35	0.98					
$A_{sw} = N_{sw1}/(D_r - D_{ro})^2$	690				470	890	235		
$A_c = q_{cl}/(D_r - D_{ro})^2$	490				620	710			
$C_{sw} = A_{sw}/A_{us}$	210				225 ~ 313	424 ~ 593	112 ~ 157	(215)	(615)
$C_c = A_c/A_{us}$	150				300 ~ 413	338 ~ 473			

N.B.) The values of D_r and D_{ro} used for deriving the values of A_{us} , A_{sw} and A_c are in ratio and not in percentage.

Fig. 2. Grain size distributions of soils, (a) traixial tests and (b) chamber tests

$$\frac{S_{us}}{\sigma'_{1c}} = A_{us}(D_r - D_{ro})^2, \quad (2)$$

where D_{ro} is the initial offset of the relative density, and D_r is in ratio and not in percentage. It is found in Fig. 3(a) for Toyoura clean sand that there would be no offset of the relative density in this relation, and it can be determined as $D_{ro}=0$. On the other hand, there needs to be some offset in this relation for silty sands, as shown in Fig. 3(b). The value of $D_{ro}=25\%$ is adopted for all the silty sands examined in the present study. This particular value was assumed, since the same values need to be assigned in Eqs. (2), (5) and (8) for the same soils, as described later. In addition, it is to note here that the adoption of this particular value might be restricted to the silty sands with fines content less than 25%. This offset of the relative density, D_{ro} , serves as a parameter which defines the lowest value of the relative density attainable under the presence of confining stress. The value of D_{ro} is in effect controlled by the volume compression during consolidation. Because of its compressibility, the silty sand is found to show some value of D_{ro} .

In the above Eq. (2), in taking account of the effects of effective overburden stress, the undrained shear strength was normalized with respect to the initial effective major principal stress. This would be guaranteed because any non-linearity between these two values were not detected within the range of σ'_{1c} tested. The values of A_{us} inferred for the four soils are summarized in Table 1.

Fig. 3. Plots of undrained shear strength ratio S_{us}/σ'_{1c} against relative density D_r , (a) Toyoura clean sand and (b) Omigawa silty sand

SWEDISH WEIGHT SOUNDING TEST

Swedish weight sounding test is frequently used for field inspections on road and railway embankments as well as for residential house constructions. Since it is relatively easy to handle the equipment and to carry out the field tests, this test is also useful for earthquake reconnaissance field investigations. The details of the test equipment and testing procedure are described by Tsukamoto et al. (2004b). The testing procedure consists of two phases, static and rotational penetration. In the first phase, the screw-shaped point at the tip of the steel rod weighing 5 kg (49 N) is statically penetrated into the ground by putting the weights of 2×10 kg (98 N) and 3×25 kg (245 N) to achieve the total weight equal to 100 kg (980 N), while the depth of rod penetration is measured. The value of W_{sw} is defined as the sum of the weights at each load application. In the second phase, upon holding the total weight of 100 kg (980 N), the penetration rod is rotated by using the horizontal bar fixed at its top. The number of half a turn necessary to penetrate the rod through 25 cm is counted and converted to the value of N_{sw} (ht/m).

In the present study, the penetration resistance, N_{sw} , observed during Swedish weight sounding tests is examined based on the previous study composed of a multiple series of calibration pressure chamber tests on clean fine sand and various silty sands, (Tsukamoto et al., 2004b). In what follows, the general framework and outcome of

Fig. 4. Calibration chamber (after Tsukamoto et al., 2004b)

the previous study are first described. The formulation of the penetration resistance, N_{sw} , against the overburden stress, σ'_v , and relative density, D_r , is then re-examined and rearranged in a manner compatible with the formulation of the undrained shear strength as described above.

Testing Details

The calibration pressure chamber used in the present study is 78.7 cm in diameter and 92.4 cm in depth, as shown in Fig. 4. The important feature of this apparatus is that the vertical stress and horizontal stress can be applied independently to a soil sample in the chamber by inflating the rubber membranes. The dry soil sample was poured into the chamber by the method of air pluviation and tamping, and various values of the relative density, D_r , were achieved. All the soil samples were normally consolidated with the earth pressure coefficient K_0 ranging between 0.2 and 0.3. A series of Swedish weight sounding tests were conducted under various values of the relative density, D_r , and vertical and horizontal stresses, σ'_v and σ'_h . The physical properties of the soils used in the chamber tests are summarized in Table 1. Toyoura sand is clean fine sand and all the other soils are fine sands with some amount of non-plastic fines. The grain size distributions of these soils are also shown in Fig. 2(b).

The outcome of the test results using dry soil samples in the chamber tests is later used for evaluating the undrained residual strength of saturated soils. It would be appropriate to assume in the present study that the mobilisation of shear strength in dry soil samples would in effect be equivalent to that in fully saturated soil samples, since there is no effect of pore-air and pore-water interactions under these two conditions.

Formulation

The field penetration resistance of soils is known to be affected mainly by soil density, effective overburden stress and grain composition among others, (Meyerhof, 1957; Liao and Whitman, 1986; Skempton, 1986; Ishihara, 1993, 1996; Cubrinovski and Ishihara, 1999; Tsukamoto et al., 2004b; and others). In the present study, the effects of soil density and effective overburden stress on the N_{sw} -value are formulated by adopting the same procedure as that described in the previous study, (Tsukamoto et al., 2004b). The basic procedure of the formulation is briefly described below.

It was found in the previous study that because the static penetration resistance represented by the value of W_{sw} has some influence on the rotational penetration resistance, N_{sw} , especially when the soil density is low and the overburden stress is small, it is convenient to have an offset parameter with respect to the value of N_{sw} , which is equivalent to the static resistance $W_{sw} = 100 \text{ kg}$ (980 N). The converted overall resistance N'_{sw} was then introduced by defining this offset as α_{sw} as follows,

$$N'_{sw} = N_{sw} + \alpha_{sw}, \quad (\alpha_{sw} = 40). \quad (3)$$

The above Eq. (3) holds true under the phase of rotational penetration, where the value of N_{sw} is larger than 1. On the other hand, N'_{sw} would be allowed to take a value less than or equal to α_{sw} under the phase of static penetration in a manner that $N'_{sw} = \alpha_{sw} W_{sw}$ (kN). For example, it would be $N'_{sw} = 20$ at $W_{sw} = 0.5 \text{ kN}$.

It then follows that the value of N'_{sw} may be normalized with respect to the square root of vertical stress to eliminate the effects of overburden stress. The value of N'_{sw1} may therefore be defined as follows,

$$N'_{sw1} = N'_{sw} \sqrt{\frac{\sigma'_o}{\sigma'_v}} = N'_{sw} \sqrt{\frac{98}{\sigma'_v}}, \quad (4)$$

where σ'_o is taken as 98 kPa and σ'_v is in kPa. The above Eq. (4) was verified by confirming the linear relation between N'_{sw} and $\sqrt{\sigma'_v}$, (Tsukamoto et al., 2004b). The linear relation between N'_{sw1} and D_r was then approximately assumed in the previous study. However, from the viewpoint of establishing the relation between N_{sw} and the undrained shear strength ratio as formulated in Eq. (2), it would be preferable to normalize the value of N'_{sw1} in a manner similar to Eq. (2). The plots of the values of N'_{sw1} against D_r were reproduced for the four soils as shown in Fig. 5, and the following relation was also found to work well,

$$N'_{sw1} = A_{sw}(D_r - D_{r0})^2, \quad (5)$$

where D_r is in ratio and not in percentage. It is to note here that the same values of D_{r0} are assumed in Eqs. (2) and (5), where $D_{r0} = 0$ for Toyoura sand and $D_{r0} = 25\%$ for all the other silty sands. The values of A_{sw} inferred for the four soils are summarized in Table 1. Herein, the data for Toyoura clean sand were obtained under the relative density ranging from $D_r = 30\%$ to 70% , as shown in Fig. 5. The data for the other silty sands were obtained under the relative density larger than $D_r = 50\%$. This is due to

Fig. 5. Plots of converted values of N'_{sw1} against relative density D_r

the fact that it was almost technically difficult to prepare dry soil samples with low density in a large chamber. Since the outcome of the laboratory triaxial tests indicated that the borders between “flow” and “no flow” for Toyoura clean sand and for the other silty sands are given at about $D_r = 30\%$ and 60% , respectively, the data of the chamber tests are rather restricted to dense soil samples. It is nonetheless assumed that the expression given in Eq. (5) can be extrapolated reasonably well.

It is to note here that in case of Swedish weight sounding tests, the chamber size effects would be negligible compared with cone penetration tests as discussed later, and are not taken into account.

CONE PENETRATION TEST

The electric cone penetrometer has been commonly used for field investigations. It electrically measures the tip resistance and shaft friction as well as pore pressure by employing the loadcell and strain-gauged transducers encased in the cone tip, and provides a continuous profile of soil layers. The evaluation of liquefaction potential of soils has been examined by establishing the correlation between the cone tip resistance and cyclic resistance of soils, (Robertson and Campanella, 1985; Seed and De Alba, 1986; Shibata and Teparaska, 1988; and others).

In what follows, based on multiple series of calibration chamber tests conducted for the present study, the cone tip resistance, q_c , is formulated against the overburden stress, σ'_v , and relative density, D_r , in a manner compatible with the formulation of the undrained shear strength as described above.

Testing Details

The calibration chamber used for a series of cone penetration tests is the same as that for Swedish weight sounding tests and is shown in Fig. 4. The soil samples used for the cone penetration tests are Toyoura sand, Omigawa sand No. 2 and Inage sand, as summarized in Table 1. The same procedure as described above for

Swedish weight sounding tests is employed in preparation of dry soil samples in the calibration chamber. The soil samples with three different values of relative density, D_r , are prepared, and are then subjected to different combinations of vertical stress σ'_v and horizontal stress σ'_h , where σ'_v changes from 30 to 100 kPa, and K_o ($=\sigma'_h/\sigma'_v$) changes from 0.4 to 1.5.

The type of cone penetrometer employed in the present study is commonly used in practice, and has an area of 10 cm² at its tip and an apex angle of 60 degrees. The cone penetrometer is then pushed into the soil sample in the chamber by means of a hydraulic actuator at a constant loading rate of 1 cm/s. The tip resistance measured in the chamber tends to increase with depth and achieves peak at depths of around 40 to 60 cm from the top surface, and such peak values of the tip resistance measured in the chamber are denoted as q_{cc} hereafter in the present study.

Chamber Size Effects

It was commonly accepted in the past studies that the cone tip resistance would be normalized with respect to the square root of vertical stress, (Jamiolkowski et al., 1988; Tatsuoka et al., 1990). and there would be a linear relation in the logarithmic plots of relative density, D_r , against $q_{cc}/\sqrt{\sigma'_v}$, as follows,

$$D_r = -a + b \log \left(\frac{q_{cc}}{\sqrt{\sigma'_v}} \right). \quad (6)$$

Such plots for Toyoura sand, which are obtained from the results of the present study, are shown in Fig. 6, where the data are fairly in good agreement with the relation for Toyoura sand obtained by Tatsuoka et al. (1990). In the study by Tatsuoka et al. (1990), the K_o -value was changed from about 0.3 to 0.5 depending upon its void ratio.

It is known that the boundary condition and chamber size exhibit important effects on the measured values from cone penetration tests, (Schnaid and Houlsby, 1991; Salgado et al., 1998; Jamiolkowski et al., 2001; and

others). It is rather difficult to consider explicitly the effects of boundary condition and chamber size. It was suggested by Jamiolkowski et al. (2001) that given the diameter ratio of $R_d = D_c/d_c$, where D_c is the diameter of the chamber and d_c is the diameter of the cone tip, the value of cone tip resistance, q_{cc} , measured in the chamber can be converted to the value of cone tip resistance, q'_c , corresponding to the free field condition equivalent to $R_d = D_c/d_c = 100$. The value of R_d imposed in the present study can be determined as $R_d = D_c/d_c = 78.7 \text{ (cm)}/3.57 \text{ (cm)} = 22.1$. It was suggested that the conversion can be implemented by the expression of $q'_c = c \times q_{cc}$, where c is the conversion parameter, which was given as $c = 0.054 \times D_r^{0.827}$ for Toyoura sand with the relative density greater than $D_r = 34.1\%$ under $R_d = 22.1$. The value of c is in effect the ratio of field to chamber penetration resistance, and is greater than 1 at the relative density greater than $D_r = 34.1\%$ due to what is called the chamber size effects exerted by highly dilatant soil samples such as Toyoura sand. The conversion parameter would be dependent upon the grain composition of soils. However, it remains to be resolved. Based on the study by Salgado et al. (1998), as the diameter ratio R_d changes from 25 to 120, the ratio of chamber to field penetration resistance would vary between 0.5 to 0.9 in case of heavily dilatant samples such as silica clean sand, while it would vary between 0.8 to 0.98 in case of compressive samples. It would therefore be reasonable to assume that the field cone penetration resistance for compressive silty sands tends to be overestimated when the same conversion parameter as Toyoura clean sand was assumed. However, for the sake of simplicity, this conversion is applied for all the soil samples used in the present study.

Formulation

It has commonly been accepted to normalize the cone tip resistance with respect to the square root of vertical stress, as can be seen in Eq. (6). Herein, in formulating the cone tip resistance, the dimensionless parameter, q'_{cl} , is introduced as follows,

$$q'_{cl} = \frac{q'_c}{\sigma'_o} \sqrt{\frac{\sigma'_o}{\sigma'_v}} = \frac{q'_c}{\sqrt{\sigma'_o \sigma'_v}} = \frac{q'_c}{\sqrt{98 \sigma'_v}}, \quad (7)$$

where σ'_o is taken as 98 kPa, and q'_c and σ'_v are in kPa. It then follows that the values of q'_{cl} can be plotted against the relative density, D_r , as shown in Figs. 7(a), (b) and (c). Also shown in these diagrams are the relations given as follows,

$$q'_{cl} = A_c (D_r - D_{r0})^2, \quad (8)$$

where D_r is in ratio and not in percentage. It is to note here that the above Eq. (8) is defined in the same form as in Eq. (5). It is found that Eq. (8) can represent this relation reasonably well, as shown in Figs. 7(a), (b) and (c), though there seem to be some effects of K_o in the case of Omigawa and Inage silty sands. Here the values of D_{r0} assumed for Eq. (8) are the same as those for Eq. (5). The values of A_{sw} inferred for the three soils are also summarized in Table 1. Herein, the data for Toyoura clean sand

Fig. 6. Logarithmic plots of values of $q_{cc}/\sqrt{\sigma'_v}$ against relative density D_r (Toyourea sand)

Fig. 7. Plots of corrected values of $q_c / \sqrt{98\sigma'_v}$ against relative density D_r , (a) Toyoura sand, (b) Omigawa sand and (c) Inage sand

were obtained under the relative density ranging from $D_r = 30\%$ to 70% , as shown in Fig. 7(a). The data for the other silty sands were obtained under the relative density larger than $D_r = 70\%$, as shown in Figs. 7(a) and (b). This is due to the fact that it was almost technically difficult to prepare dry soil samples with low density in a large chamber. Since the outcome of the laboratory triaxial tests indicated that the borders between “flow” and “no flow” for Toyoura clean sand and for the other silty sands are given at about $D_r = 30\%$ and 60% , respectively, the data of the chamber tests are rather restricted to dense soil samples. It is nonetheless assumed that the expression given in Eq. (8) can be extrapolated reasonably well.

EVALUATION OF UNDRAINED SHEAR STRENGTH

The undrained shear strength ratio was defined as given in Eq. (1) by normalizing the undrained shear strength, S_{us} , with respect to the initial effective major principal stress, σ'_{1c} . However, in usual circumstances except for the situations where soil densification is employed for liquefaction countermeasures, the initial effective vertical stress, σ'_{vo} , is nearly equivalent to σ'_{1c} . Therefore, it would be reasonable to assume that the undrained shear strength ratio, S_{us}/σ'_{1c} can be replaced by S_{us}/σ'_{vo} . In addition, it was described above that the undrained shear strength ratio, S_{us}/σ'_{1c} ($=S_{us}/\sigma'_{vo}$), Swedish penetration resistance, N_{sw} , and cone tip resistance, q_c , can be formulated with respect to the relative density, D_r , in the same manner as each other, as seen in Eqs. (2), (5) and (8). By taking advantage of such compatible formulations and eliminating the effects of D_r , the correlations of S_{us}/σ'_{vo} with N_{sw} and q_c can be derived as follows,

$$\frac{S_{us}}{\sigma'_{vo}} = \frac{N'_{sw1}}{C_{sw}} = \frac{N_{sw} + 40}{C_{sw}} \sqrt{\frac{98}{\sigma'_{vo}}}, \quad (9)$$

$$\frac{S_{us}}{\sigma'_{vo}} = \frac{q_{c1}}{C_c} = \frac{q_c}{C_c \sqrt{98\sigma'_{vo}}}, \quad (10)$$

where the parameters for vertical stress, σ'_v , included in Eqs. (4) and (7) were all replaced by σ'_{vo} . Herein, C_{sw} and C_c are the parameters uniquely determined for any given soil, and $C_{sw} = A_{sw}/A_{us}$ and $C_c = A_c/A_{us}$. It is to note here that the cone tip resistance corresponding to the free field is denoted as q_c in Eq. (10). The more robust and versatile correlations between the undrained shear strength and field penetration resistance have long been examined based on case histories (Ishihara et al., 1990a; and others). Such direct correlations can be derived as follows,

$$S_{us} = \frac{N'_{sw}}{M_{sw}} = \frac{N_{sw} + 40}{M_{sw}}, \quad M_{sw} = \frac{C_{sw}}{\sqrt{98\sigma'_{vo}}}, \quad (11)$$

$$S_{us} = \frac{q_c}{M_c}, \quad M_c = C_c \sqrt{\frac{98}{\sigma'_{vo}}}, \quad (12)$$

where σ'_{vo} is in kPa. M_{sw} and M_c are the conversion parameters, where M_{sw} is in per kPa and M_c is dimensionless. These conversion parameters are found to be dependent upon the level of vertical stress. The values of C_{sw} and C_c are estimated for the soils used in the present study, as shown in Table 1. Herein, the value of A_{us} for Toyoura sand is assumed as $A_{us} = 3.3$, and those for the other silty sands are assumed to range from 1.5 to 2.1. The values of C_{sw} and C_c are then calculated by dividing the values of A_{sw} and A_c by the value of A_{us} thus estimated for each soil.

The values of C_{sw} and C_c are found to vary among the soils, as shown in Table 1. Since the state parameters influencing the soil behaviour, such as soil density and overburden stress, were incorporated in the formulation and the effects of such state parameters were eliminated,

Fig. 8. Plots of values of C_{sw} against void ratio range $e_{max}-e_{min}$

Fig. 9. Plots of values of C_c against void ratio range $e_{max}-e_{min}$

the values of C_{sw} and C_c would only be dependent upon the grain composition of soils. Some of the recent studies have shown that the void ratio range, $e_{max}-e_{min}$, serves as a good parameter in representing the grain composition of soils for characterizing the behaviour of silty sands, (Miura et al., 1997; Cubrinovski and Ishihara, 1999, 2000a, 2000b). The values of C_{sw} and C_c inferred from the present study are therefore plotted against the void ratio range, as shown in Figs. 8 and 9. The void ratio range for clean sand is generally lower than 0.4, and it becomes larger as the fines content increases in the soil. Figures 8 and 9 suggest that the largest values of C_{sw} and C_c can be found when the soil contains some fines. This would imply according to Eqs. (11) and (12) that the soils containing some fines exhibit the lowest undrained shear strength when the same values of N_{sw} or q_c are observed at a given depth in the clean sand deposit and silty sand deposit.

As a typical example, the correlations for Toyoura sand are illustrated in Figs. 10 and 11 in the plots of the undrained shear strength ratio, S_{us}/σ'_{vo} , against the Swedish penetration resistance, N_{sw} , and the cone tip resistance, q_c . Since it was found in the previous studies that the threshold value of S_{us}/σ'_{ic} ($=S_{us}/\sigma'_{vo}$) dividing the conditions of “flow” and “no flow” can be uniquely determined ranging typically from 0.24 to 0.26 among silty sands, (Tsukamoto et al., 2004a, 2008), the ranges of

Fig. 10. Relation between values of W_{sw} and N_{sw} against undrained shear strength ratio S_{us}/σ'_{vo}

Fig. 11. Relation between values of q_c against undrained shear strength ratio S_{us}/σ'_{vo}

values of N_{sw} and q_c indicative of soil layers susceptible to flow deformation can also be determined as shown in Figs. 10 and 11.

CASE HISTORY STUDIES ON SWEDISH WEIGHT SOUNDING TESTS

The post-earthquake field reconnaissance investigations were carried out at two locations where the rapid landslide and the large flow failure of fluidized subsurface soils occurred recently. In what follows, the relation between the Swedish penetration resistance and undrained shear strength of soils is examined from these two case history studies. Swedish weight sounding tests were conducted at the soil layers identical to those failed during the earthquakes.

Tsukidate Landslide

In the event of Miyagiken-oki Earthquake, which occurred on May 26, 2003, the rapid landslide occurred at Tsukidate, Miyagi, Japan, on a gentle slope consisting of reclaimed soil deposits for agricultural purposes (Uzuoka et al., 2005). The location of Tsukidate is shown in Fig. 12. The reclaimed soil deposits of 40 metres in width, 80 metres in length and 5 metres in depth collapsed and

Fig. 12. Location of site of landslide at Tsukidate

Fig. 14. Grain size distributions of soils at Tsukidate and Tanno

(a) Plan view

(b) Cross section A-A'

Fig. 13. Post-earthquake plan view and cross section of site of landslide at Tsukidate (a) plan view and (b) cross section

Fig. 15. Results of Swedish weight sounding test at Tsukidate

flowed downstream on a gentle slope with 7 degrees inclination, as shown in Figs. 13(a) and (b). The soil used for reclamation was from pyroclastic origin with pumice tuff. The physical properties of the soil are summarized in Table 1, and the grain size distribution of the soil is shown in Fig. 14. A series of Swedish weight sounding tests were carried out at this site, and the results of the test conducted at the original intact soil deposit close to the top portion of the collapsed area are indicated in Fig. 15. It is found that the Swedish penetration resistance at the possible sliding plane located at around 5.5 metres below the ground surface ranges from 0.5 kN to 0.75 kN. The undrained shear strength ratio, S_{us}/σ'_{vo} , of soils mobilized at this site is estimated to be about 0.12, based on the following simple expression (Ishihara et al., 1990a),

$$\frac{S_{us}}{\sigma'_{vo}} \approx \frac{S_{us}}{\gamma_t H} = \cos \alpha \sin \alpha, \quad (13)$$

where H and α are the depth and angle of subsurface sliding surface, and the effective vertical stress σ'_{vo} is assumed as the overburden stress $\gamma_t H$. The value of S_{us}/σ'_{vo} thus estimated is also confirmed from the results of laboratory undrained triaxial compression tests conducted on the soil sample retrieved from this site and field density measurement. The parameters inferred from this case history

Table 2. Summary of data from case history studies

Site	H (m)	α (degree)	S_{us} (kPa)	q_c (kPa)	W_{sw} (kN)/ N_{sw} (ht/m)	Reference
Tsukidate ⁱ⁾	5.5	7	12		0.5 (kN) ~ 0.75 (kN)	
Tanno ⁱⁱ⁾	4.5	3	3.5		0.25 (kN) ~ 1 (kN)	
Mochikoshi No. 1 tailings dam ⁱⁱⁱ⁾	6	8	15	200 ~ 400		Ishihara et al. (1990a)
Mochikoshi No. 2 tailings dam ⁱⁱⁱ⁾	6	9.5	17.5			
Hokkaido tailings dam ^{iv)}	4	5	7	200 ~ 300		
				2000 ~ 5000		
Metoki ^{iv)}	(2)	(?)	6	450 ~ 600		
Chonan ^{v)}	1.5 ~ 2	18	8	450 ~ 600		
Sharara ^{vi)}	10	9	13.5	—		
Firma ^{vi)}	7	13	14.5	—		
May 1 ^{vi)}	10	12	8.0	300 ~ 600		Ishihara et al. (1990b)
				500 ~ 1100		
Okuli ^{vi)}	15	0.5	2.0	100 ~ 400		
				500 ~ 900		

N.B.1) i) Miyagiken-oki (May 26, 2003), ii) Tokachi-oki Earthquake (September 26, 2003), iii) Izu-Oshima-Kinkai Earthquake (January 14, 1978), iv) Tokachi-oki Earthquake (May 16, 1968), v) Chibaken-Toho-oki Earthquake (December 17, 1987), vi) Tajikistan Earthquake (Gissar, January 23, 1989)

N.B.2) H : average depth, α : average slope angle, S_{us} : undrained shear strength, q_c : cone tip resistance, W_{sw}/N_{sw} : Swedish penetration resistance.

N.B.3) “Metoki” was the road embankment slump failure, and it was difficult to estimate the values of H and α .

Fig. 16. Location of site of landslide at Tanno **Fig. 17.** Post-earthquake plan view and cross section of site of landslide at Tanno (a) plan view and (b) cross section

study are summarized in Table 2.

Tanno Flow Failure

In the event of Tokachi-oki Earthquake which occurred on September 26, 2003, the fluidization and subsidence of gently sloped farming fields took place in Tanno area of Kitami in Hokkaido. The location of the site is indicated in Fig. 16. The farming field of 35 metres wide and 150 metres long subsided due to the eruption of fluidized subsurface deposits through a couple of ejection holes located downstream portions of the subsided area. It was found from the interview of the local people that the subsided area had been reclaimed with the deposits of local volcanic soil for agricultural purposes. The plan view and cross section of the site are shown in Fig. 17. It was found from the site survey that the bottom surface of

the fluidized subsurface deposits forms a round basin in a transverse cross section, and the fluidized deposits flowed down swinging leftwards and rightwards along the basin until they were erupted at the downstream portions of the subsided area. The physical properties of the soil are summarized in Table 1, and the grain size distribution of the soil is shown in Fig. 14. A series of Swedish weight sounding tests were carried out at this site, and the results of the test conducted at the original intact soil deposit close to the top portion of the collapsed area are indicated in Fig. 18. It is found that the Swedish penetration resistance at the fluidized layer located at around 3 to 5.5 metres below the ground surface ranges from 0.25 kN to 1.0 kN. The undrained shear strength ratio, S_{us}/σ'_{vo} , of

Fig. 18. Results of Swedish weight sounding test at Tanno

soils mobilized at this site which can be estimated from Eq. (13) is estimated to be as low as 0.05. The value of S_{us}/σ'_{vo} thus estimated is also confirmed from the results of laboratory undrained triaxial compression tests conducted on the soil sample retrieved from this site and field density measurement.

Swedish Penetration Resistance

Based on the data derived from the case history studies as described above, the plots of values of S_{us}/σ'_{vo} against values of N'_{sw1} are produced as shown in Fig. 19. It is to note here that the values of N'_{sw} are determined in a manner that N'_{sw} is equal to 40 at $W_{sw} = 1.0$ kN. For example, N'_{sw} is equal to 20 at $W_{sw} = 0.5$ kN. The parameter C_{sw} can then be inferred as the ratio of N'_{sw1} to S_{us}/σ'_{vo} . It is to note here that the parameter C_{sw} is considered to be only dependent on the grain composition of soils, since the effects of overburden stress and soil density are eliminated during the course of its formulation. The values of C_{sw} thus calculated are superimposed on the diagram correlating the parameter C_{sw} with the void ratio range, $e_{max}-e_{min}$, as shown in Fig. 20. The plot for the site of Tsukidate is located within the range of its correlation derived from the laboratory study conducted in the present study. However, the plot for the site of Tanno is out of range probably because of its extremely low value of undrained shear strength. It is to note here that this extremely low value was derived based on the average slope angle using Eq. (13). However, as described above, the fluidized deposits seem to have flowed down swinging leftwards and rightwards along the basin from bottom to top portions, indicating that the local failure slope angle might be greater than the average angle, due to such basin effects.

CASE HISTORY STUDIES ON CONE PENETRATION TESTS

A number of case history studies were reported by Ishihara et al. (1990a) on the flow failures of tailings dams during the past earthquakes in Japan. A series of large-

Fig. 19. Plots of values of S_{us}/σ'_{vo} against N'_{sw1} from case history studies

Fig. 20. Plots of values of C_{sw} against void ratio range $e_{max}-e_{min}$ from case history studies

scale flow failures of wind-lain collapsible loess deposits in Tajikistan were also reported by Ishihara et al. (1990b), which occurred during the earthquake on January 23, 1989. In the present study, the relation between the cone tip resistance and undrained shear strength of soils is examined based on the data of undrained shear strength of soils estimated from each site of flow failure and the results of Dutch cone penetration tests conducted during the post-earthquake field reconnaissance investigations. The details of the case history studies reported by Ishihara et al. (1990a, 1990b) are summarized in Table 2. The main focus of the past studies was to estimate the undrained shear strength of soils from post-earthquake stability analysis, and to correlate it with the cone tip resistance observed along the depth of 'collapsed' debris of sliding soil mass. Since the present study is focused on the cone tip resistance of pre-earthquake 'intact' soil deposits, the values of cone tip resistance observed at the 'intact' portions of the soil deposits, typically located immediately below the failure surfaces, are read off from the diagrams reported in these past studies.

The values of S_{us}/σ'_{vo} are then plotted against the values

Fig. 21. Plots of values of S_{us}/σ'_{vo} against q_{c1} from case history studies, (a) 'collapsed' layers and (b) 'intact' layers

of q_{c1} observed at 'collapsed' layers of soil deposits, as shown in Fig. 21(a). The same plots are produced for the values of q_{c1} , which are read off from the 'intact' layers, as shown in Fig. 21(b). Since the parameter C_c can be inferred as the ratio of q_{c1} to S_{us}/σ'_{vo} , the possible ranges of C_c -values for loess and ordinary silty sand can be calculated from the data shown in Figs. 21(a) and (b). The values of C_c thus calculated are superimposed on the diagram correlating the parameter C_c with the void ratio range, $e_{max}-e_{min}$, as shown in Fig. 22. Herein, the values of $e_{max}-e_{min}$ for the silty sands and loess are assumed as 0.5 and 0.7. The values of C_c for 'collapsed' layers are found to be extremely lower than those for 'intact' layers. This would be reasonable since the 'collapsed layers' correspond to the loosely deposited debris that collapsed due to seismic shaking, flowed down and finally came to rest just in equilibrium with its stability. The values of C_c for 'intact' layers estimated from the case history studies are found to be within the range of its correlation derived from the laboratory study conducted in the present study.

CONCLUSIONS

The undrained shear strength ratio of silty sands was formulated with respect to the relative density, based on

Fig. 22. Plots of values of C_c against void ratio range $e_{max}-e_{min}$ from case history studies

the outcome of the previous studies on laboratory triaxial tests. The penetration resistances of Swedish weight sounding tests and cone penetration tests were formulated with respect to the effective overburden stress and relative density, based on laboratory calibration chamber tests. By combining these formulations, the correlations of the undrained shear strength with Swedish penetration resistance and cone tip resistance were established. The example charts for evaluating the undrained shear strength from Swedish and cone penetration resistances for Toyoura sand were shown, in which the possible ranges of values of penetration resistances indicative of soil layers which would be susceptible to flow deformation were also indicated. The correlations of the undrained shear strength of soils with field penetration resistances were then examined based on a number of case history studies, and were found to work well.

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NOTATION

- a, b : parameters in Eq. (6)
- c : conversion parameter for chamber-size effects
- d_c : diameter of cone tip
- q_c (kPa): cone tip resistance
- q_{cc} (kPa): cone tip resistance measured in calibration chamber
- q_{c1} : converted cone tip resistance: $= q_c / \sqrt{98\sigma'_v}$, (unit in kPa)
- q'_c (kPa): cone tip resistance corrected for calibration chamber size effects

A_c : parameter correlating converted cone tip resistance and relative density

$$: = q'_{c1}/(D_r - D_{ro})^2$$

A_{sw} : parameter correlating converted N_{sw} -value and relative density: $= N'_{sw1}/(D_r - D_{ro})^2$

A_{us} : parameter correlating undrained shear strength ratio and relative density

$$: = (S_{us}/\sigma'_{1c})/(D_r - D_{ro})^2 \doteq (S_{us}/\sigma'_v)/(D_r - D_{ro})^2$$

C_c : parameter correlating converted cone tip resistance and undrained shear strength ratio

$$: = q'_{c1}/(S_{us}/\sigma'_{1c}) = A_c/A_{us}$$

C_{sw} : parameter correlating converted N_{sw} -value and undrained shear strength ratio

$$: = N'_{sw1}/(S_{us}/\sigma'_{1c}) = A_{sw}/A_{us}$$

D_c : diameter of chamber

D_{ro} : constant determining offset of relative density

M_c : parameter correlating cone tip resistance and undrained shear strength

$$: = q_c/S_{us} = C_c\sqrt{98/\sigma'_{vo}}, \text{ (unit in kPa)}$$

M_{sw} (per kPa): parameter correlating N'_{sw} -value and undrained shear strength

$$: = N'_{sw}/S_{us} = C_{sw}/\sqrt{98\sigma'_{vo}}, \text{ (unit in kPa)}$$

N'_{sw} : $N'_{sw} = N_{sw} + 40$ at $N_{sw} \geq 1$ and $N'_{sw} = \alpha_{sw}W_{sw}$ (kN) at $W_{sw} \leq 1$ kN

N'_{sw1} : $= N'_{sw}\sqrt{98/\sigma'_{vo}}$, (unit in kPa)

$$R_d = D_c/d_c$$

α_{sw} : $= 40$: offset parameter against N_{sw} taking into account the effects of static penetration

σ'_o : reference effective stress: $= 98$ kPa

Abbreviated words

TC: triaxial compression

TE: triaxial extension

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