SOILS AND FOUNDATIONS Vol. 48, No. 1, 101–114, Feb. 2008
Japanese Geotechnical Society

EFFECTS OF CLAY CONTENT ON LIQUEFACTION CHARACTERISTICS OF GAP-GRADED CLAYEY SANDS

WEN-JONG CHANG[i] and MING-LIN HONG[ii]

ABSTRACT

A series of undrained, cyclic simple shear tests were performed on reconstituted specimens with various clay contents to study the effects of clay content on liquefaction characteristics of clayey sands based on a framework of an idealized binary packing model and intergrain state parameters. From observed liquefaction characteristics, clayey sands with different clay contents can be grouped as sand-like or clay-like soils depending on the clay content and the transitional fines content of the sand-clay mixture. A simple equation is derived and verified to correlate the transitional fines content with the void ratios of the clean sand and the pure clay consisting of the mixture. In addition, a new relationship for clay content correction is proposed based on the linear relationship between the cyclic resistance ratio and the clay content at the same intergranular void ratio. The cyclic resistance ratio of sand-like clayey sands can be divided into two components: (1) the resistance of the sand skeleton at the specific intergranular void ratio, and (2) the increment of cyclic resistance from clayey fines. The rate of increment for cyclic resistance varies with the properties of contained clay particles. Data from three independent studies have shown the proposed procedure is promising.

Key words: clayey sand, cyclic resistance ratio, intergranular void ratio, liquefaction, transitional fines content (**IGC**: D7)

INTRODUCTION

The fines content, defined as soil portion finer than No. 200 sieve (0.074 mm), in sandy soils has been recognized as one of the major factors affecting the liquefaction resistance of granular soils (e.g., Seed et al., 1985; Ishihara and Koseki, 1989). Most natural and manmade liquefiable deposits contain certain amount of silt and/or clay. According to Lade and Yamamuro (1997), soils with fines are the most common soil type in observed liquefied sites. Generally, nonplastic or low plasticity silty fines are more common in land stratum and gain more interest in liquefaction research. However, sands mixed with various percentages of clay, generally defined as particles finer than 0.002 mm (Mitchell and Soga, 2005), are widespread in estuaries, tailing dams, and sometimes in offshore foundations. These soils are generally termed as clayey sands (Georgiannou et al., 1991) or clayey silts. Depending on the amount of clay, liquid limits (LL), and plasticity index (PI), these soils are classified as SC or double symbol soils (e.g., SP-ML or SP-CL) in Unified Soil Classification System (USCS). Liquefactions of clayey sands subjected to cyclic loading were reported in offshore sites (e.g., Tjelta et al., 1988) and near-shore reclaimed sites (Ishihara et al., 1981). Recently, many liquefied sites in central-west Taiwan during the 1998 Chi-Chi earthquake contain significant portions of silty and clayey fines (Huang et al., 2004). To properly evaluate the liquefaction resistances of clayey sands, understanding the effects of clay content in clayey sands under undrained cyclic loading is warranted.

Although the term "liquefaction" has been historically used to describe a variety of phenomena that involve soil deformations caused by static or dynamic loading under undrained conditions, the generation of excess pore water pressure plays a key role in all liquefaction-related phenomena. For loose and saturated sands under undrained shearing, the volumetric contractions among granular soil particles are retarded, resulting in generations of positive excess pore pressure. Therefore, significant volumetric contraction is needed to initiate liquefaction. Since the amount of volumetric strain is mainly affected by the relative density and the mean effective stress level of a given soil (Seed, 1979), the relative density (or void ratio) has been considered as one of the major factors affecting the normalized liquefaction resistance, represented as cyclic resistance ratio ($CRR = \tau/\sigma'_v$, where τ is the cyclic shear stress to initiate liquefaction and σ'_v is the initial vertical effective stress of the soil). For a clean granular soil, the global void ratio (e) can well represent the "average" packing condition of the particles. However, for a sand-fines mixture, the presence of the fines can significantly influence the microstructure/fabric of the mixture, resulting in sig-

[i] Assistant Professor, Dept. of Civil Engrg, National Chi Nan University, Taiwan (wjchang@ncnu.edu.tw).
[ii] Former Graduate Student, ditto.
The manuscript for this paper was received for review on May 7, 2007; approved on September 20, 2007.
Written discussions on this paper should be submitted before September 1, 2008 to the Japanese Geotechnical Society, 4-38-2, Sengoku, Bunkyo-ku, Tokyo 112-0011, Japan. Upon request the closing date may be extended one month.

nificantly different shearing behaviors. Different fine content corrections have been developed and implemented in engineering practice for evaluations of liquefaction potential (e.g., Youd et al., 2001; Bray and Sancio, 2006).

The effects of fine particles on undrained shearing characteristics of mixed soils depend on the grading of the sand particles, amount of fines, mineralogy of the fines, and distributions of fines within the mixture (Thevanayagam and Martin, 2002). For sand-clay mixtures, the clayey fines will affect the void distribution and the plasticity properties of soils (Mitchell and Soga, 2005). Therefore, Atterberg limits and clay percentage by weight had been used for screening of liquefaction vulnerability (Andrews and Martin, 2000). Guo and Prakash (1999) reviewed test data on undisturbed and reconstituted silt-clay specimens and concluded that clayey fines in low plasticity mixtures increase the rate of pore pressure generation due to reduction in hydraulic conductivity, resulting in decrements of the cyclic resistance. However, the cyclic resistance increases for high plasticity mixtures because of the imparting of the soil cohesion and the minimum liquefaction resistance occurred at $PI=4$. Boulanger and Idriss (2004) suggested that the granular soils with fines can be divided into sand-like and clay-like soils. The cyclic stress ratios against liquefaction or cyclic failure are greater for clay-like soils and the boundary for the two soil types is at $PI=7$ for engineering practice. Sanin and Wijewickreme (2006) conducted direct simple shear tests on undisturbed natural silts and found that ''cyclic mobility'' type failure occurred in silts and the CRR of silts increases with increasing OCR. In summary, current studies in correlating the liquefaction resistance with PI in clay-sand mixtures show inconsistencies and further study is needed.

Alternatively, intergrain state concept, which considers the microstructure of particle packing, has been proposed to explain the effects of nonplastic fines in granular soils (Lade and Yamamuro, 1997). Ueng and Chang (1982) mixed fine silica sand with kaolinite to study the effects of clay content on liquefaction behavior of clayey sands. In their study, relative density of the sand structure, which is the relative density of sands ignoring the clay, was used to prepare reconstituted specimens. Their results show that the clay content will increase the cyclic resistance but decrease the induced shear strain amplitude at the initiation of liquefaction for specimens with the same relative density of the sand structure.

An idealized binary packing model that describes the intergrain state of two-size particle mixtures is implemented in this study to simplify the complicated packing system. An experimental program was conducted to systematically study the cyclic liquefaction characteristics of clayey sands. Clean sand is mixed with kaolinite to prepare reconstituted specimens at different void ratios and clay contents. A series of stress-controlled, undrained, cyclic direct simple shear (CDSS) tests, which is recognized as a better laboratorial testing technique compared with cyclic triaxial testing in liquefaction study in terms of representing the earthquake loading conditions, were performed to determine the cyclic liquefaction resistance under K_0 condition. The effects of clay contents on stress-strain relationships, characteristics of pore pressure generation, and variations in cyclic resistances are discussed. Comparisons of liquefaction resistance with different void parameters are performed to assess the validation of the intergrain state concept. Clayey sands are divided into sand-like or clay-like soils based on the clay content. Furthermore, the CDSS testing results and other previous published data are compiled to develop a new representation of clay content correction for evaluating the liquefaction resistance of sand-like clayey sands.

INTERGRAIN STATE PARAMETERS

Void Parameters of Binary Packing Model

In natural soil deposits, the packing condition and void distribution are complicated due to the large variations in particle size and deposition conditions. In order to rationally characterize the packing condition of mixed soils, binary packing models have been proposed. The three major assumptions in an idealized binary packing model are: (1) only two particle sizes are considered, (2) the diameter ratio between the coarse and the fine particle is large, and (3) the packing of the coarse particles is not affected by the fine particles and vice versa. Gap-graded soils (i.e., soils with gap grain size distribution) satisfy the first two assumptions and hence are more appropriate for this framework.

Different binary packing models have been implemented to describe the engineering properties of mixed soils, such as the strength of sandy gravels (Fragaszy and Siddiqi, 1992), undrained shear strength of silty sands (Georgiannou et al., 1990) and liquefaction resistance of silty sands (Kuerbis et al., 1988; Polito and Martin, 2001). As void distribution is one of the major factors affecting the shearing behavior of gap-graded soils at intermediate to large strain levels, different void parameters inferred from idealized binary packing models have been used to correlate with the cyclic resistances of silty and clayey sands.

The global void ratio (e), defined as the volumetric ratio between the summations of all voids and the volume of all soil particles, has been used as a packing index for soils with relatively uniform size (e.g., clean sands or pure clay). For sand-clay mixtures, the global void ratio varies with clay content, as illustrated in Fig. 1. In Fig. 1, the global void ratios of the clean sand and the pure clay are e_1 and e_2 respectively and e_2 is generally greater than e_1. For sand-clay mixtures with low clay content (i.e., the A-B section in Fig. 1), the clay particles are contained within the voids formed by sand particles and the global void ratio decreases as the clay content increases. The engineering behaviors of those low clay content soils are primarily dominated by the contact conditions of the sand particles. Conversely, for mixtures with high clay content (i.e., the B-C section), the sand particles are dispersed by the clay and the soil behaviors are mainly con-

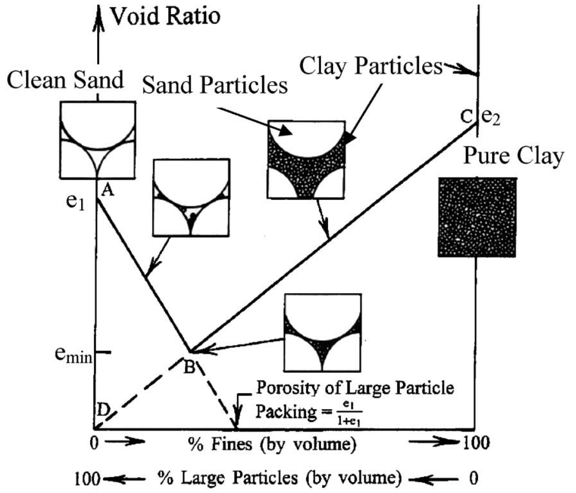

Fig. 1. Variation of global void ratio in binary packing structure (after Lade et al., 1998)

trolled by the contact conditions of the clay particles. At point B, the voids formed by sand particles are completely filled with clay and the global void ratio is the minimum. At this stage, the clay-sand mixture is at a metastable condition with the lowest strength (Lade et al., 1998).

For clayey sand, the packing condition of the sand particles is characterized by the intergranular void ratio (e_s) (Kuerbis et al., 1988), defined as:

$$e_s = (e + \mathrm{CC})/(1 - \mathrm{CC}) \qquad (1)$$

where e is the global void ratio, CC is the clay content defined as soil portion finer than 0.002 mm by weight and expressed as decimal. Similarly, the packing condition of clay particles is characterized by the interfine void ratio (e_f), defined as:

$$e_f = e/\mathrm{CC} \qquad (2)$$

Depending on the amount of clay content, engineering properties of the mixed soils are dominated by either sand or clay particles, resulting in sand-like or clay-like soil behaviors respectively. For instance, soils with low clay contents where clay particles are contained within the voids formed by sand particles, the engineering properties of the mixed soils are primarily controlled by the packing properties of the sand particles. Consequently, the intergranular void ratio is a more representative packing index to correlate sand-like soil properties affected by the packing voids. Conversely, for high clay content soils, the interfine void ratio is more representative to correlate clay-like soil properties. The specific clay content that divides the mixed soils into sand-like or clay-like soils is called the transitional fines content (denoted as TFC) (Thevanayagam and Martin, 2002). Implementing the clay content and TFC, the sand-clay mixtures can be systematically classified as either sand-like or clay-like soils and the packing conditions are characterized with the intergranular or interfine void ratios accordingly.

Derivation of Transitional Fines Content

As aforementioned, the TFC is a useful classification index for liquefaction screening and other geotechnical applications. In the binary packing framework, the metastable packing condition divides the mixture to sand-like or clay-like soils as well. By assuming that the TFC occurs at the metastable packing condition and importing the non-interference assumption (assumption (3)) in the binary packing model, the TFC can be correlated with the void ratios of the clean sand and pure clay.

Assume the void ratios of the clean sand and the pure clay under a specific effective stress are e_1 and e_2 (as denoted in Fig. 1) respectively. As no interference occurs between the sand and clay particles, the void ratios of the sand and the clay particles at metastable condition remain e_1 and e_2 respectively. For a unit volume of sand particles (i.e., $V_s = 1$ volumetric unit) the total volume of voids formed by the sand particles is e_1. At metastable condition, all the voids formed by sand particles are occupied by the clay particles packed at a void ratio of e_2. Therefore, the summation of the volumes of clay particles (V_f) and the volume of voids (V_v) is equal to e_1, or expressed as:

$$V_f + V_v = e_1 \qquad (3a)$$

Using the definition of void ratio of the clay particles, the volume of clay particles (V_f) and the volume of voids are correlated by:

$$\frac{V_f}{V_v} = \frac{1}{e_2} \qquad (3b)$$

Solving Eqs. (3a) and (3b), the volume of clay particles and the volume of voids are expressed as:

$$V_f = \frac{e_1}{V_s + e_2} = \frac{e_1}{1 + e_2} \qquad (4a)$$

$$V_v = \frac{e_1 e_2}{1 + e_2} \qquad (4b)$$

The mass of sand particles and clay particles is G_s and $G_f V_f$, where G_s and G_f are the specific gravities of the sand and clay particles respectively. The clay content at metastable condition ($\mathrm{FC_B}$) is:

$$\mathrm{FC_B} = \frac{V_f G_f}{V_s G_s + V_f G_f} = \frac{(e_1/(1+e_2))G_f}{G_s + (e_1/(1+e_2))G_f} \qquad (5)$$

For natural soils, values of G_s and G_f are very close and can be assumed the same. Recall that TFC is assumed approximately equal to $\mathrm{FC_B}$; the TFC of a sand-clay soil at a specific effective confining stress can be expressed as:

$$\mathrm{TFC} = \frac{e_1}{1 + e_1 + e_2} \qquad (6)$$

Eq. (6) demonstrates that factors affecting the void ratios of clean sand and pure clay at a specific confining stress will vary the range of TFC if non-interference condition is valid. It should be noted that the value of e_1 is within the maximum and minimum void ratios of the clean sand and e_2 should be corrected to the mean effective stress.

Thevanayagam and Martin (2002) further considered the interference cases by assuming that certain amount of clay particles are in contact with the sand particles and proposed that the equivalent intergranular void ratio $(e_s)_{eq}$ can be calculated from:

$$(e_s)_{eq} = [e + (1-b)CC]/[1-(1-b)CC] \tag{7}$$

where b is the percentage of clay in contact with the sand particles with respect to all clay particles in the mixed soil. The above expression simply subtracts the amount of clay particles in contact with the sand particles and ignores the void change due to these clay particles. Accordingly, the expression of the TFC in Eq. (6) is modified as:

$$\mathrm{TFC} = \frac{1}{(1-b)} \frac{e_1}{1+e_1+e_2} \tag{8}$$

Traditionally, the global void ratio has been considered as a packing index to correlate the cyclic resistance for sand-clay mixtures. However, on the basis of a binary packing model, the intergranular void ratio and interfine void ratio should be used to correlate with the cyclic resistances and other engineering properties for sand-like or clay-like soils respectively. Although these void parameters have been implemented to depict the effects of silts on undrained shear strength and cyclic resistance of silty sands, no systematic study has been conducted in clayey sands. In addition, gap-graded clayey sand fits better with the major assumptions of an idealized binary packing model. A systematic study is performed and presented to verify the adequateness of intergrain state concept in clayey sands and eventually a new approach for clay content corrections on cyclic resistance of clayey sands is developed based on the intergrain framework.

TESTING PROGRAM

Cyclic Direct Simple Shear Apparatus

A modified NGI-type cyclic direct simple shear (CDSS) testing system, which had been used to study pore pressure generation characteristics of silty sands (Hazirbaba and Rathje, 2004), is used in this study. Details of the CDSS testing system are shown in Fig. 2. Two hydraulic actuators are used for the normal and shear loading/deformation control and a pneumatic actuator for cell pressure control. The three close-looped actuators are controlled by a Window-Based digital controller, which enables the system to perform cyclic tests under various loading conditions including stress-controlled, strain-controlled, and constant volume-controlled shearing. The soil specimen, shearing frame, and normal loading frame are sealed in a pressure chamber that allows for imposing back-pressure and cell pressure. To minimize the rocking deformation of soil specimens due to lack of the complementary shear stresses on the vertical side, the normal loading frame has been reinforced by a high stiffness supporting system. Additionally, pre-stressed sliding rollers are used in both normal and shear loading mechanisms to minimize and maintain constant frictions. The frictions measured from the shearing and normal loading mechanisms are 14 N and 5 N respectively. These values are used for correcting shear stress calculations and imposing the compensation stress in the normal direction.

In this study, all specimens are 68 mm in diameter and 25 mm in height. To maintain K_0 condition throughout the consolidation and shearing stages, a wire-reinforced latex membrane was used to confine the lateral deformation of the specimen. In order to assure that no sliding occurred between the caps and the soil specimen, the cap surfaces contacted with the specimen are roughed to increase friction. Circular porous stones are embedded at

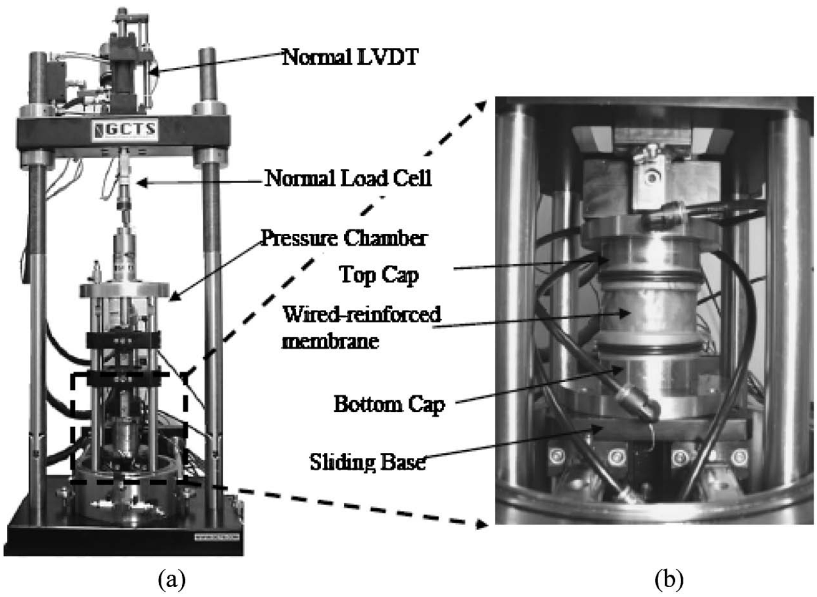

Fig. 2. Details of CDSS testing system: (a) components of CDSS system and (b) setup of specimen

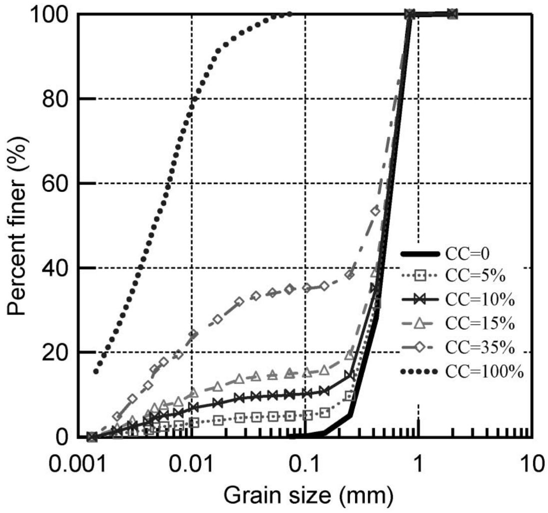

Fig. 3. Grain size distribution of testing soils

the center of the caps to provide drainage paths for pore pressure measurement and back pressure application.

Sample Preparation and Testing Conditions

Reconstituted specimens of clean sands mixed with kaolinite had been used to study the undrained cyclic behavior of clayey sands (e.g., Ishihara and Koseki, 1989; Georgiannou et al., 1991). In this study, a fine silica sand retained on #200-sieve was used as the clean sand and a commercially available kaolinite is used as the fines. The fine silica sand is originally from Vietnam and had been used in various liquefaction studies in Taiwan for its consistent gradation and easy purchasing (Ueng et al., 2006). The Vietnam sand is white in color with a specific gravity of 2.65, maximum void ratio of 0.92, and minimum void ratio of 0.61. The Vietnam sand is classified as poorly graded sand (SP) in Unified Soil Classification System (USCS). The kaolinite is also white in color and has a specific gravity of 2.61, liquid limit of 49%, and plasticity index of 19%. The USCS classification of the kaolinite is clayey silts with slight plasticity (ML). The grain size distributions of the Vietnam sand and kaolinite are shown in Fig. 3. The D_{50} for the clean Vietnam sand and the kaolinite is 0.5 mm and 0.005 mm respectively. The uniform size distribution and large diameter ratio between the sand and clay particles agrees with the first and second assumptions of the binary packing model.

Previous studies (e.g., Vaid et al., 1999; Ishihara, 1996) had revealed that sample preparation method could significantly affect the cyclic liquefaction resistance of the reconstituted granular specimens. Although the water sedimentation procedure described in Ishihara (1996) and the slurry deposition method used by Kuerbis et al. (1988) are generally considered to be closer to the natural deposition process, they are either too complicated to be performed in a limited space or too difficult to control void ratios. Furthermore, particle segregation will be more significant in sand-clay mixtures than the sand-silt mixtures due to the larger size difference between particles. Therefore, moist tamping procedure in controlled volume fashion was used in this study to prepare specimens with uniformly distributed clay at designated void ratios. Weighted oven-dried sands and kaolinite were mixed with 5% water first and divided into two equally weighted portions. The first half was placed into the wired reinforced membrane fixed on the bottom cap and tamped to half of the specimen height (12.5 mm). Before the placement of the second portion of the mixture, the soil surface was roughened to a depth of about 2 mm. Subsequently, the other half was placed and tamped to full height of the specimen (25 mm). As the height and diameter of all specimens are the same for all specimens, the void ratio and clay content of each specimen can be controlled by changing the weights of the sand and kaolinite.

The saturation process was performed by flushing deaired water into the specimen with a 10 kPa vacuum pressure on top of the specimen, followed by applying a back pressure of 95 kPa and a cell pressure of 100 kPa for at least 30 minutes. The saturation was verified by the B-value test and a specimen with B-value greater then 0.95 is considered saturated. After saturation, a 150 kPa effective vertical stress (σ'_v) was applied in the normal direction and the cell pressure was maintained at the saturation stage (100 kPa). Since the lateral side was confined by a wired reinforced membrane, a K_0-consolidation is achieved. The height of the consolidated specimen was measured to calculate the consolidated void ratio and the shear strain during shearing.

To determine the undrained cyclic resistance of specimens under K_0 conditions, the consolidated specimens were subjected to a constant-amplitude, sinusoidal shear stress at a loading frequency of 0.1 Hz. During the shearing stage, a constant volume condition is achieved by fixing the height of the specimen and closing the drainage paths. The testing conditions represent the undrained behavior of a level-ground soil element subjected to upward propagating shear waves. In addition, the combination of wired reinforced membrane and constant volume condition maintained the K_0 conditions during cyclic loading to some extent. The cyclic shearing was terminated when initial liquefaction was achieved. Initial liquefaction is said to be achieved when the excess pore pressure ratio ($r_u = \Delta u/\sigma'_v$, Δu = excess pore pressure) exceeds 0.9 or the double shear strain amplitude (DA) becomes greater than 6%. The first criterion is referred from Hazirbaba and Rathje (2004) and the second one is adopted from Ishihara (1996). The differences between the two failure criteria will be verified and presented later.

Testing Program

The testing program consists of 4 testing series with clay contents of 0%, 5%, 15% and 35%. Additionally, 3 specimens with void ratio of 0.61 and clay content of 10% were tested specifically to construct the relationship between cyclic resistance ratio and clay content. The USCS classifications for specimens with clay contents of 5%, 10%, 15%, and 35% are SP-SM, SP-SM, SM, and SM respectively. The grain size distributions of all testing

Table 1. Summary of testing program and results

Sample Number	Consolidated Void Parameter			CSR[1]	N_{ru}[2]	N_{DA}[3]	Sample Number	Consolidated Void Parameter			CSR	N_{ru}	N_{DA}
	e	e_s	e_f					e	e_s	e_f			
CC0–1	0.79	0.79	—	0.17	11	9	CC10–1	0.61	0.79	6.1	0.16	76	73
CC0–2	0.79	0.79	—	0.15	25	24	CC10–2	0.60	0.78	6.0	0.19	26	24
CC0–3	0.8	0.8	—	0.13	47	46	CC10–3	0.61	0.79	6.1	0.21	8	7
CC0–4	0.8	0.8	—	0.11	258	258	CC15–1	0.79	1.11	5.3	0.11	78	76
CC0–5	0.76	0.76	—	0.15	60	58	CC15–2	0.775	1.088	5.2	0.13	24	21
CC0–6	0.76	0.76	—	0.17	22	20	CC15–3	0.78	1.1	5.2	0.15	7	5
CC0–7	0.76	0.76	—	0.21	6	5	CC15–4	0.766	1.077	5.1	0.10	175	173
CC0–8	0.69	0.69	—	0.21	14	12	CC15–5	0.76	1.07	5.1	0.11	67	65
CC0–9	0.71	0.71	—	0.20	24	23	CC15–6	0.772	1.085	5.2	0.13	14	11
CC0–10	0.68	0.68	—	0.19	42	40	CC15–7	0.71	1.01	4.7	0.13	72	69
CC0–11	0.69	0.69	—	0.18	149	142	CC15–8	0.7	1	4.7	0.15	11	8
							CC15–9	0.71	1.01	4.7	0.18	3	2
CC5–1	0.78	0.88	15.6	0.11	117	116	CC35–1	0.79	1.76	2.3	0.14	—	50
CC5–2	0.78	0.87	15.6	0.15	12	11	CC35–2	0.8	1.77	2.3	0.15	—	11
CC5–3	0.79	0.89	15.9	0.17	5	4	CC35–3	0.8	1.77	2.3	0.17	—	2
CC5–4	0.75	0.85	15.1	0.13	79	78	CC35–4	0.74	1.68	2.1	0.16	—	66
CC5–5	0.75	0.84	15.1	0.15	26	25	CC35–5	0.75	1.69	2.1	0.18	—	15
CC5–6	0.76	0.85	15.2	0.17	9	8	CC35–6	0.74	1.68	2.1	0.19	—	8
CC5–7	0.7	0.79	14.0	0.17	113	110	CC35–7	0.72	1.64	2.1	0.17	—	53
CC5–8	0.7	0.79	14.0	0.18	36	34	CC35–8	0.73	1.66	2.1	0.18	—	39
CC5–9	0.71	0.8	14.2	0.19	13	11	CC35–9	0.71	1.63	2.0	0.19	—	7

Note: 1. CSR: cyclic stress ratio
2. N_{ru}: number of cycles to induce $r_u = 0.9$
3. N_{DA}: number of cycles to induce double strain amplitude of 6%

series are shown in Fig. 3. The clay contents of the 4 testing series are adopted from the boundary values in the empirical correlation of cyclic resistance with corrected SPT-*N* values from Seed et al. (1985). Also, the designated 4 clay contents intended to cover the TFC range of the mixtures; therefore, sand-like and clay-like soil behaviors can be observed. To study the effects of void parameters, specimens were prepared to global void ratios of 0.8, 0.75, and 0.7 in each test series. For each combination of clay content and void ratio, at least 3 specimens were tested. The time histories of the shear stress, shear strain, and excess pore pressure ratio were calculated from recorded shear forces, shear deformations, and excess pore pressures respectively. The relationship between the cyclic stress ratio ($CSR = \tau/\sigma'_v$, where τ is the applied shear stress amplitude) and the number of cycles to reach initial liquefaction was developed to represent the cyclic resistance of the soil specimen subjected to different earthquake magnitudes.

The testing program and results are summarized in Table 1. All specimens are named according to the clay content by weight followed by the testing sequence. For example, CC0–1 represents the first specimen for clean sand (CC = 0) series. There are 11 tests for the clean sand series, 9 tests for the other three series, and 3 tests for CC = 10%. The global void ratio, intergranular void ratio, and interfine void ratio of each specimen are computed using the consolidated specimen height and Eqs. (1) and (2). The applied cyclic stress ratios and the number of cycles to induce excess pore pressure ratio of 0.9 and double strain amplitude of 6% are listed in the same table. More details are presented in the following sections.

TESTING RESULTS AND SYSTEM COMPLIANCE

Testing Results and Data Reduction

For specimens with clay contents of 0%, 5%, 10%, and 15%, the undrained cyclic shearing behaviors are similar in terms of stress-strain relationship and pore pressure generation pattern. Significant differences were observed for specimens with a clay content of 35%. Typical testing results for a clean sand specimen (CC = 0%) and specimens with the highest kaolinite content (CC = 35%) are shown in Figs. 4(a) and 4(b) respectively. The low clay content specimens behaved like loose clean sands and the high clay specimens behaved like dense sands under undrained cyclic shearing.

The imposed cyclic stress ratios and the number of cycles to generate excess pore pressure ratio of 0.9 and to induce double shear strain amplitude (denoted as DA) of 6% are listed in Table 1. The cyclic resistance curves, which correlate the applied cyclic stress ratios and the number of cycles to liquefaction, for the same void ratio specimens are shown in Fig. 5. Using the cyclic resistance curves, the cyclic stress ratio corresponding to liquefaction at 15 loading cycles is defined as the cyclic resistance ratio for earthquakes of magnitude 7.5 and denoted as $CRR_{7.5}$ hereafter (Seed et al., 1985). The procedure to determine the $CRR_{7.5}$ for a given soil is illustrated in Fig. 5.

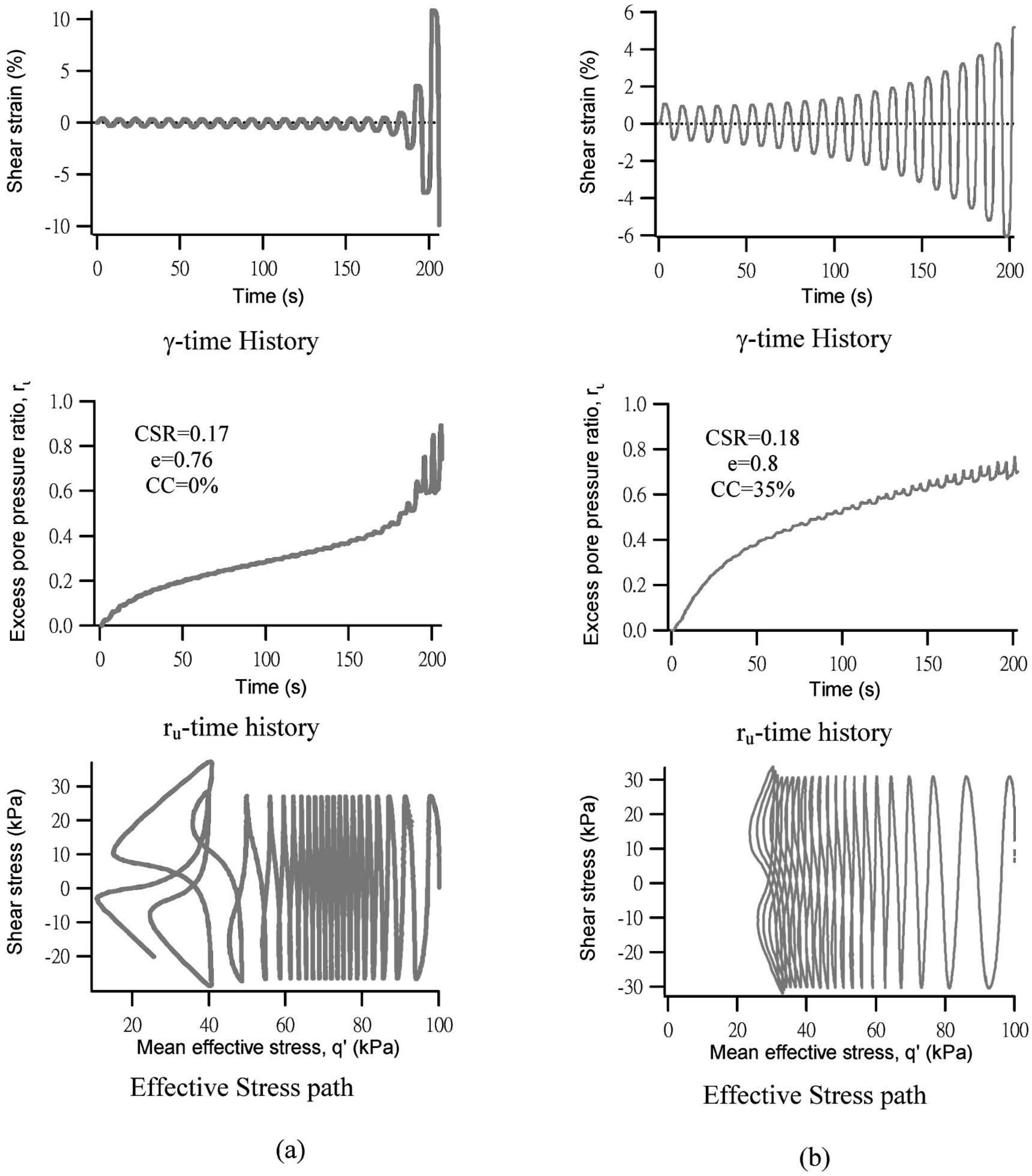

Fig. 4. Shear strain-time histories, pore pressure buildup records, and effective stress paths: (a) clean sand specimen (CC0-6) and (b) high kaolinite content specimen (CC35-2)

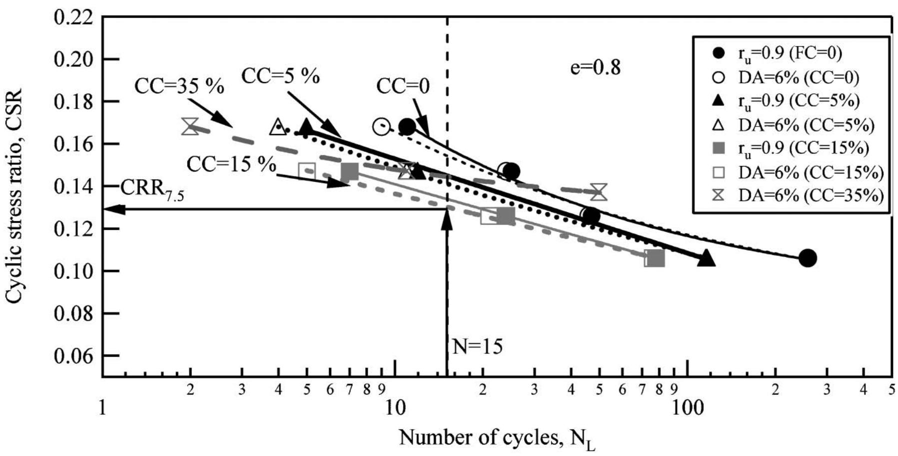

Fig. 5. Comparisons of different liquefaction criteria ($e = 0.8$)

Table 2. Summary of cyclic resistance ratios

Clay Content (CC)	Consolidated Void Parameter			$CRR_{7.5}$[1]	Specimens
	e	e_s	e_f		
0	0.79	0.79	—	0.158	CC 0–1～ 4
0	0.76	0.76	—	0.178	CC 0–5～ 7
0	0.69	0.69	—	0.208	CC 0–8～11
5	0.78	0.88	15.6	0.145	CC 5–1～ 3
5	0.75	0.85	15.1	0.158	CC 5–4～ 6
5	0.7	0.79	14.0	0.187	CC 5–7～ 9
10	0.61	0.79	6.1	0.21	CC10–1～ 3
15	0.78	1.1	5.2	0.134	CC15–1～ 3
15	0.77	1.08	5.2	0.125	CC15–4～ 6
15	0.7	1.01	4.7	0.142	CC15–7～ 9
35	0.80	1.77	2.3	0.144	CC35–1～ 3
35	0.74	1.68	2.1	0.177	CC35–4～ 6
35	0.72	1.65	2.1	0.183	CC35–7～ 9

Note: 1. $CRR_{7.5}$: cyclic resistance ratio against magnitude 7.5 earthquake

Variations in Criteria of Initial Liquefaction

For clean sands under undrained cyclic shearing, initial liquefaction is defined as either 100% of excess pore pressure ratio is generated or the induced strain level is significantly large. The state of $r_u = 100\%$ generally occurred at the proximity of 3.75% of shear strain amplitude for clean sands (Ishihara, 1996). However, for sands with fines, the buildup of r_u can be only up to 90 to 95% when significant strain is induced (Ishihara, 1996). In this study, both $r_u = 90\%$ and DA = 6% are used as the termination criteria. The cyclic stress ratios and loading cycles to achieve the two criteria for specimens with void ratio of 0.8 are shown in Fig. 5. For specimens with clay contents less than 15%, both criteria can be reached and the variations in number of cycles to achieve the two criteria are small, resulting in less than 3% variations in $CRR_{7.5}$ defined by both criteria. The small variations agree with the previous studies from clean sands (Ishihara, 1996). However, only the double shear strain amplitude of 6% was developed for specimens with clay content of 35% and the generated excess pore pressure ratios were less than 80%. The absence of high excess pore pressure ratios for specimens with clay content of 35% is due to different failure mechanisms. To be consistent, the values of $CRR_{7.5}$ presented in the following sections are determined based on the DA = 6% criteria and these values are tabulated in Table 2.

EFFECTS OF CLAY CONTENT ON LIQUEFACTION CHARACTERISTICS

Stress-Strain Relationship

The effects of clay content on stress-strain relationship are illustrated from hysteretic loops. The hysteretic loops of specimens with similar void ratio but different clay content are shown in Figs. 6(a) ～ 6(d). Again, the stress-strain relationship of the clayey sand with clay content of 35% was distinct from the others. For specimens with clay content less than 15%, constant shear strain amplitudes with small amount of strain accumulation was induced before the occurrence of initial liquefaction and sharp shear strain increments occurred around the occurrence of initial liquefaction. A collapsed-type failure behavior, which was generally observed in loose clean sands, occurred in low clay content specimens. This outcome indicates that the undrained cyclic shearing was mainly controlled by the packing of sands. For specimens with 35% clay content, the induced shear strain amplitude constantly increased and a progressive failure type, which was often observed in dense clean sands or clay, was observed. Therefore, the undrained shearing of clayey sands with high clay content is dominated by the clay particles.

The secant shear moduli from the first loading cycle, denoted as G_1, are also shown in Fig. 6. Comparison of the shear moduli of the first loading cycle indicates that G_1 decreases as the clay content increases under the same shear strain level. For specimens with clay content of 35%, the value of G_1 is the smallest and the reduction of G_1 is much significant. The variation between the clay content and G_1 indicates that the soil stiffness at small to medium strain levels decreases as clay content increases.

Excess Pore Pressure

Effects of clay content on characteristics of pore pressure generation are shown in terms of rate of pore pressure generation. The normalized rates of excess pore pressure generation for specimens of same void ratio but different clay contents are shown in Fig. 7, along with the typical band of experimental data form triaxial and cyclic simple shear tests on sands (Seed et al., 1975). The results show that the patterns of pore pressure generation are different between sands with clay content less than 15% and sands of 35% clayey fines. For sands with clay content less than 15%, the rate of pore pressure generation is relatively constant for excess pore pressure ratio below 0.6, followed by a rapid increase of pore pressure generation, which is consistent with the collapsed-type failure behavior. In comparison with the typical band compiled by Seed et al. (1975), the collapsed-type failure mechanism is more significant for Vietnam sands. In addition, comparisons among specimens with clay content less than 15% shows that the clay content has only minor influence on the rate of pore pressure generation.

On the contrary, a faster rate of pore pressure generation is observed for sands with 35% clay content for r_u less than 0.4. A slower but constant rate of pore pressure generation for r_u exceeding 0.4 is observed. The pore pressure generation pattern agrees with the observed accumulation of shear strain prior to liquefaction.

Cyclic Resistance Ratio

Using DA = 6% as the criteria for initial liquefaction to construct cyclic resistance curves, the values of $CRR_{7.5}$ for a specific void ratio and clay content was evaluated and the values are listed in Table 2. The relationships among the cyclic resistance ratio ($CRR_{7.5}$), global void ratios (e), and clay contents (CC) are developed from Table

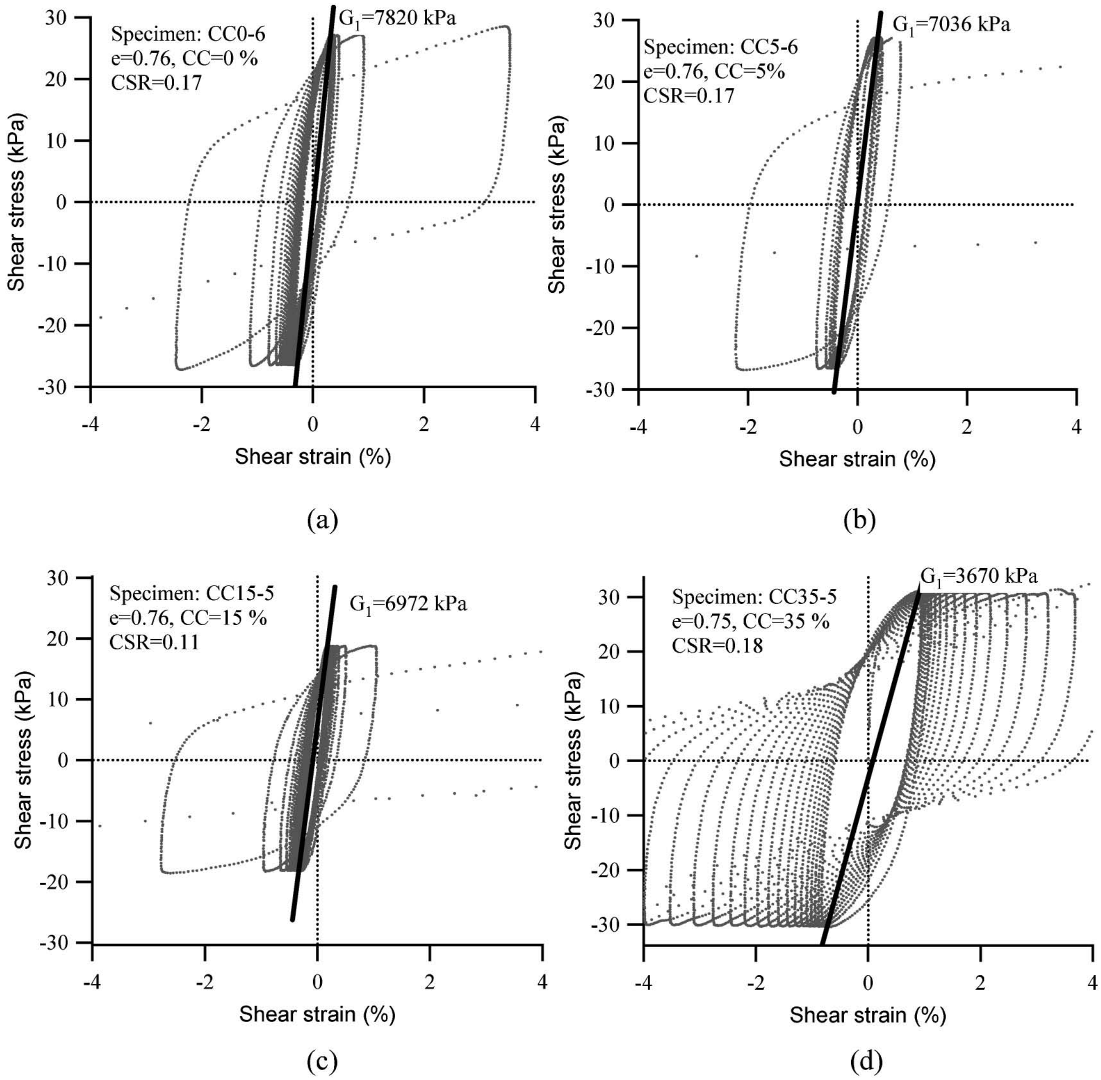

Fig. 6. Hysteretic loops for sands with different clay contents: (a) clean sand (CC0–6), (b) CC = 5% (CC5–6), (c) CC = 15% (CC15–5) and (d) CC = 35% (CC35–5)

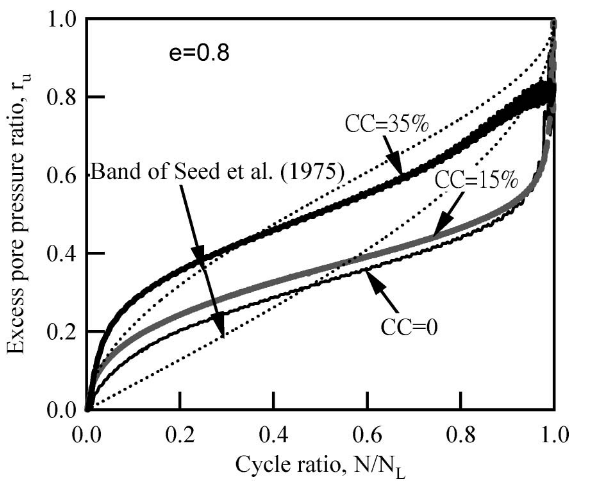

Fig. 7. Comparisons of rates of excess pore pressure generation

2 and shown in Fig. 8. The variations of $CRR_{7.5}$ with global void ratio at the same clay content are shown in Fig. 8(a). The results show that the $CRR_{7.5}$ decreases linearly as the global void ratio increases for clayey sand with the same clay content. The variations of $CRR_{7.5}$ at different clay content but the same void ratio are shown in Fig. 8(b). No monotonic relationship exists between the $CRR_{7.5}$ and clay content in clayey sands prepared at the same void ratio. For specimens with clay content less than 15%, the cyclic resistance ratio decreases with increasing clay content. The trend reverses for specimens with 35% clay content. This trend of variation is similar to the results by Polito and Martin (2001) for non-plastic silty sands except that the fines content corresponding to the minimum $CRR_{7.5}$ is different.

Based on the stress-strain relationships, patterns of excess pore pressure generation and variations of cyclic resistance ratio with clay content, the tested clayey sands can be clearly divided into two distinct soil types. The boundary of clay content for classification is within 15% to 35%. More details will be discussed on the framework of intergrain state and presented later.

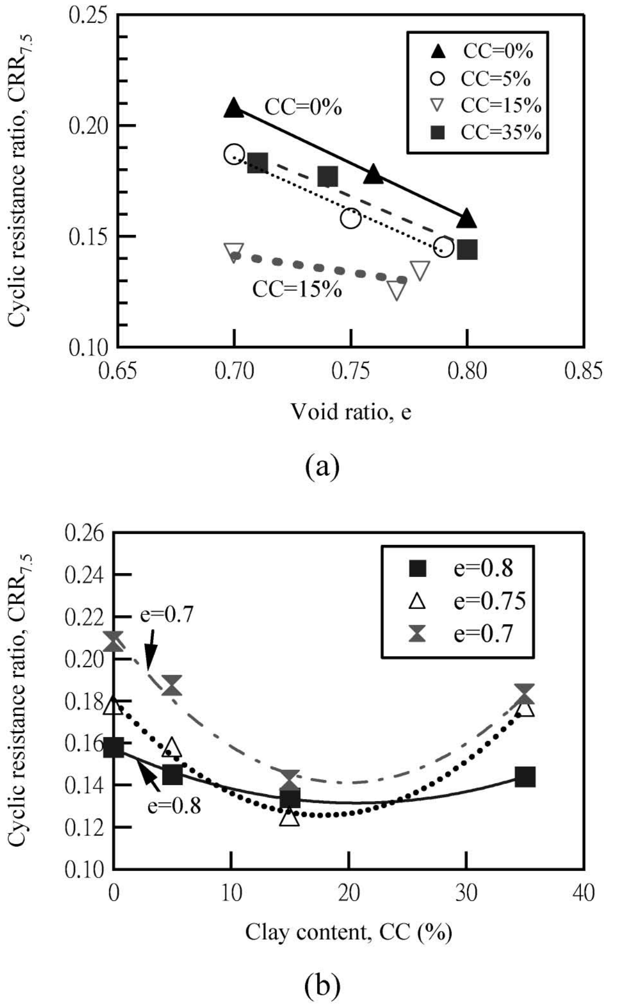

Fig. 8. Variation of cyclic resistance ratio with clay content and global void ratio *e*: (a) variations between $CRR_{7.5}$ and *e* for different CC and (b) relationship between $CRR_{7.5}$ and CC for different *e*

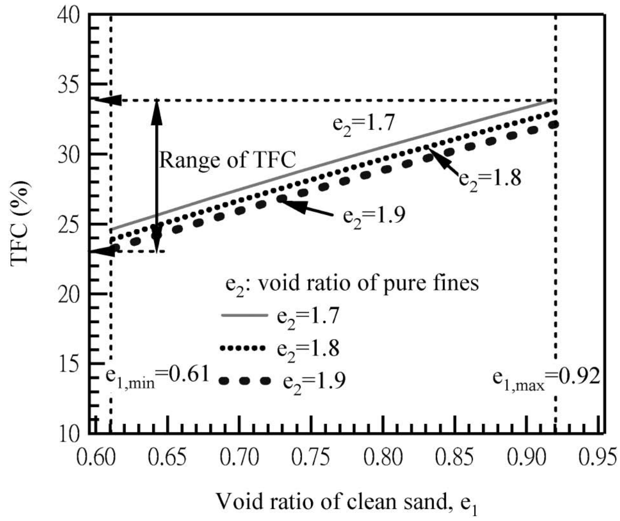

Fig. 9. Predicted range of TFC for Vietnam sand with kaolinite at a mean effective stress of 100 kPa

INTERPRETATIONS BASED ON BINARY PACKING MODEL

Verification of Predicted TFC

Based on the testing results, sands with clay contents less than 15% and 35% behaved differently, indicating that the TFC for these sand-kaolinite mixtures falls between 15% and 35%. Previous studies by Lade et al. (1998), Polito and Martin (2001), and Cubrinovski and Ishihara (2002) show that TFC for natural and gap-graded soils is between 20% and 35%. The range of void ratio for kaolinite under a mean effective stress of 100 kPa is from 1.7 to 1.9 (Mitchell and Soga, 2005). The maximum and minimum void ratios of the Vietnam sand are 0.92 and 0.61 respectively. Assuming that the *b* value in Eq. (8) is 0 (i.e., no clay particle in contact with the sands), the variations of the TFC are shown in Fig. 9 and the TFC ranges from 17% to 26%. The predicted range basically agrees with the testing results and the outcome supports that the proposed relationship is a rational approach.

With regards to the binary packing model framework, sand-clay mixtures can be divided into sand-like and clay-like soils depending on the clay content and TFC of mixtures. The rational agreement of the predicted TFC will be beneficial for soil categorization. Accordingly, specimens of this sand-clay mixture with clay content less than 15% behave as sand-like soils and specimens with clay content of 35% or higher behave as clay-like soils. Since liquefaction is more prone in sand-like soils, the following discussions on cyclic resistance of clayey sand will focus on results from clay content less than 15%.

Cyclic Resistance Ratio and Intergrain State Parameter

As aforementioned in lecture review and relationships between $CRR_{7.5}$ and CC, no consistent conclusion had been made using global void ratio as the packing index. Alternatively, intergranular void ratios, which represent the packing condition of sand/coarse particles in sand-clay mixtures, are used to correlate with cyclic resistance ratios and clay/fine contents. The relationship between the cyclic resistance ratios, intergranular void ratios, and clay contents are presented in Fig. 10. In Fig. 10(a), the $CRR_{7.5}$ decreases linearly as the intergranular void ratio (e_s) increases for specimens with the same clay content. This trend implies that loose packing of sand particles produces smaller cyclic resistance against liquefaction no matter how many clay particles are mixed. This trend is also consistent with the relationship between $CRR_{7.5}$ and global void ratio (*e*).

To highlight the contribution of the clay content to cyclic resistances in sand-like mixtures, the variation of $CRR_{7.5}$ with CC at a constant intergranular void ratio of 0.79 is plotted, as shown in Fig. 10(b), along with silty sand results by Polito and Martin (2001). The discussions will be limited to soils with CC less than TFC because the cyclic resistances of sand-like soils are more practical and these soils have similar liquefaction mechanisms. For CC $=0.15\%$ and $e_s=0.79$, the void ratio is 0.52, which is smaller than the minimum void ratio of clean Vietnam

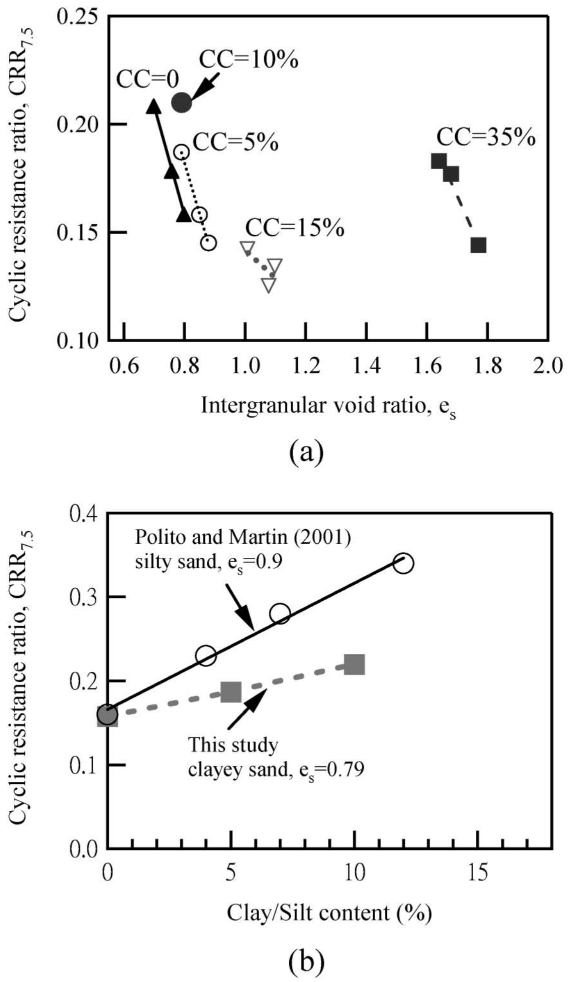

Fig. 10. **Variation of $CRR_{7.5}$ with FC and intergranular void ratio: (a) variation of $CRR_{7.5}$ with e_s with the same CC and (b) variation between $CRR_{7.5}$ and FC at a constant e_s**

sand ($e_{min} = 0.61$); therefore, it is impossible to prepare specimens satisfying this combination of CC and e_s. As a result, data for CC = 15% is excluded from the plot and CC = 10% and $e_s = 0.79$ testing series were performed to extend the applicable CC range in the plot.

Testing results for specimens with clay content less than 10% show a linear incremental relationship exists between the $CRR_{7.5}$ and clay content for sand-like clayey sands with the same intergranular void ratio. Similar trend had been reported by Polito and Martin on Yatesville sand with non-plastic silts (Fig. 10(b)). The monotonic relationship demonstrates that using the intergranular void ratio as the void index for sand-like soils is advantageous in correlating $CRR_{7.5}$ and clay content.

The positive linear relationship of cyclic resistance ratio with clay content at the same intergranular void ratio can be explained conceptually by the assumption of non-interference between sand and clay particles (assumption (3)). When a sand-clay mixture with CC less than TFC (i.e., AB region in Fig. (1)) is subjected to cyclic load, the resistance of the mixture is the summation of the resistances from sand particles and clay particles. Based on non-interference assumption, the resistances of sand particles and clay particles are generated independently. As clay content increases, more clay particles are contained within the void formed by sand particles, resulting in more clay particles being sheared generating more resistance. In conclusion, using the TFC for classification and the linear relationship between the $CRR_{7.5}$ and clay content evidently show that the binary packing framework is valid and is beneficial in characterizing undrained cyclic shearing behaviors of gap-graded soils.

Clay Content Correction for Clayey Sands

Based on the linear relationship between the cyclic resistance ratio and clay content at a constant intergranular void ratio, the cyclic resistance ratios of sand-like, clayey sands can be expressed as:

$$CRR(e_s,\ CC) = [CRR_{cs}(e_s)] + n(CC) \qquad (9)$$

where n is the rate of increment (i.e., slope of the line) between the CRR and CC for sand-like clayey sands with the same intergranular void ratio and $CRR_{CS}(e_s)$ is the intercept of the line, which represents the cyclic resistance ratio of the sand skeleton (i.e., clean sand) at an intergranular void ratio of e_s. Eq. (9) describes that the CRR in sand-like clayey sands can be divided into two components: the CRR of sand skeleton and CRR increment by clayey fines. The value of $CRR_{CS}(e_s)$ can be determined from clean sand tests with global void ratio equal to the intergranular void ratio because the global void ratio is equal to the intergranular void ratio for CC = 0. If the slope n and CC are known and the CC is less than the TFC of the clayey sand, the increment of CRR from the clay content can be computed and the CRR of sand-like soils can be predicted accordingly. It is worthwhile to address that separation of the two components in Eq. (9) is based on the non-interference assumption between the sand and clay particles in the idealized binary packing model and the adoption of intergranular void ratio to represent the packing condition of the host sand.

The $CRR_{CS}(e_s)$ represents the cyclic resistance ratio from the sand skeleton and generally is the major contributor of the cyclic resistance ratio for clayey sands. Consequently, factors affecting the CRR of clean sands, such as the relative density, soil fabric, prior seismic straining, stress history, and aging (Seed, 1979), will significantly alter the value of $CRR_{CS}(e_s)$. On the other hand, the value of n mostly depends on the mineralogy of clay fines based on the discussion of effects of fines on undrained shear characteristics of mixed soil by Thevanayagam and Martin (2002). To verify the hypothesis on n value, data from two independent studies by Ueng and Chang (1982) and Polito and Martin (2001) are compiled and compared with current study. Properties of testing materials by Ueng and Chang (1982) and Polito and Martin (2001) are listed in Table 3.

The tests by Ueng and Chang (1982) were conducted on reconstituted sand-kaolinite specimens prepared to a designated sand skeleton relative density (D_{rs}), which can be directly converted to an intergranular void ratio using the conversion between the relative density and void ra-

Table 3. Summary of compiled data

Data Set	Host sand		Fines			D_r^2 (%)	C_r^3	Reference
	void range	D_{50}^1 (mm)	D_{50} (mm)	PI (%)	USCS			
Fulung-Drs40	1.0 –0.64	0.23	0.002	29	CH	~40	0.55	Ueng and Chang (1982)
Fulung-Drs50	1.0 –0.64	0.23	0.002	29	CH	~50	0.58	Ueng and Chang (1982)
Vietnam sand	0.92–0.61	0.5	0.005	19	ML	~42	1.0	This study
Yatesville sand	0.97–0.65	0.18	0.03	NP	ML	<40	0.55	Polito and Martin (2001)

Note: 1. D_{50}: mean grain size
2. D_r: relative density
3. C_r: correction factor for CRR from laboratorial testing to the field

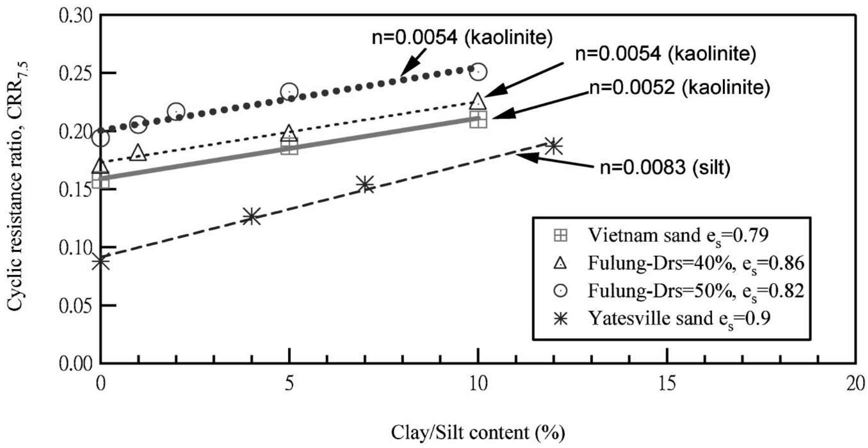

Fig. 11. Comparisons of the increments of $CRR_{7.5}$ from clay or silty fines

tio. The reconstituted specimens were isotropically consolidated under 55 kPa and cyclically sheared to liquefy using a cyclic triaxial (CTX) testing apparatus. The stress-strain curves show that the $D_{rs}=40\%$ and $D_{rs}=50\%$ testing series are sand-like soils with collapsed-type failure mechanism. These two data sets are used to verify the variation of n values for similar clay content. In contrast, the data of Yatesville sands with non-plastic silts are used to verify the difference of n values for different fines (i.e., silt vs. clay). This data was published by Polito and Martin (2001) was also tested by a CTX device.

To perform a fair comparison, all the $CRR_{7.5}$ values from isotropically-consolidated from CTX tests must be multiplied with a correction factor (denoted as C_r) counting for the different loading conditions and definition of cyclic stress ratio. Generally, the CRR determined by CDSS and CTX are related by (Seed and Peacock, 1971):

$$(CRR)_{CDSS} = C_r(CRR)_{CTX} \quad (10)$$

Several correction factors had been proposed (e.g., Seed and Peacock, 1971; Finn et al., 1971) and the values of C_r are within 0.55 to 1.0. Seed and Peacock (1971) proposed that the value of C_r varies with the relative density. The correction factors for relative densities below 40%, 60%, and 85% are 0.55, 0.61, and 0.7 respectively. These values are used to interpolate the C_r values in this study and listed in Table 3. For CDSS tests, the C_r value is a unity.

The relationship between cyclic resistance ratio and clay/silt content at a constant intergranular void ratio for each compiled set are shown in Fig. 11. Again, linear relationships exist for all data sets, which evidently support the representation of Eq. (9). The n values for different sands but mixed with similar kaolinite are almost the same and they are significantly different from the n value of sands mixed with silts. The variations of n value clearly indicate that the rate of increment in CRR is closely related to the properties of contained fines and are independent from the host sands. The independency of the n value from different host sands shows the non-interference between sand and clay particles.

In summary, the derivation and prediction of TFC, classifications of mixed soil based on CC and TFC, components of CRR, and the independency of n value clearly show that using the binary packing model can be a useful framework in characterizing the cyclic resistance behaviors. Although the validation of Eq. (9) for silty fines needs further investigation, the proposed procedure for fines content correction to cyclic resistance ratio of mixed soils is promising as long as the mixed soils satisfy the assumptions of the idealized packing model. Nevertheless, the proposed relationship could be valuable in systematically evaluating the CRR for clayey sands with clay content less than TFC.

SUMMARY AND CONCLUSIONS

Series of undrained, cyclic simple shear tests were performed to systematically study the effects of clay content in liquefaction characteristics of clayey sands. All tests were conducted on reconstituted specimens consisting of clean sands mixed with different amount of kaolinite. The idealized binary packing model and intergrain state concept are implemented to interpret the liquefaction characteristics of clayey sands. The testing results and findings are summarized as follows:

1. A simple equation is proposed to correlate the TFC with the void ratios of the clean sand and pure clay in clayey sands. The derivation is based on assumptions of an idealized binary packing model. The testing results from CDSS tests rationally agree with the proposed method.
2. The clay content in clayey sands affects the following liquefaction characteristics: (1) the stiffness at small to medium strain levels, (2) rate of pore pressure generation, (3) failure mechanism, and (4) the liquefaction resistance. For clayey sands with clay content less than TFC, the cyclic undrained behaviors are dominated by the packing conditions of the sand particles and sand-like responses are observed. However, for clayey sands with clay content exceeding TFC, the cyclic undrained behaviors are mainly controlled by the clay content and clay-like responses are observed. Accordingly, using the TFC and clay content, clayey sands can be divided into sand-like or clay-like soils.
3. Based on cyclic undrained tests on clayey sands, a linear relationship is shown to exist between the $CRR_{7.5}$ and clay content for sand-like clayey sands with the same intergranular void ratio. From this point of view, using the intergranular void ratio as the index of packing for sand-like soils is advantageous in correlating the cyclic resistance ratio and clay content.
4. A new clay content correction procedure is proposed based on the linear relationship between the cyclic resistance and clay content at the same intergranular void ratio. The cyclic resistance ratio of sand-like clayey sands can be divided into two components: the resistance of the sand skeleton at that intergranular void ratio, and the increment of cyclic resistance from clay fines. The rate of increment in cyclic resistance varies with properties of the clay particles. Compiled Data from three independent studies show that the proposed procedures are valuable in systematically evaluating the CRR for clayey sands with clay content less than TFC.

ACKNOWLEDGEMENTS

Financial support was provided by the National Science Council of ROC under grants NSC 92-2211-E-260-005 and NSC 93-2211-E-260-006. The first author would like to express sincere appreciations to Prof. Ueng at National Taiwan University for his constructive suggestions and valuable data. These supports and assistances are gratefully acknowledged.

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