INFERENCE OF BEHAVIOR OF SATURATED SANDY SOILS DURING EARTHQUAKES FROM LABORATORY EXPERIMENTS

Tzou-Shin Ueng

ABSTRACT

Laboratory experiments, including small soil element tests, centrifuge tests, and large 1 g shaking table tests have been performed for the study of sandy soil behavior before and after liquefaction. The understanding of this type of behavior is needed for development of a performance-based design for geotechnical structures under seismic loading. This paper attempts to infer the behavior of saturated sandy soils during an earthquake based on the experimental data from 1 g shaking table tests on clean sand in a large biaxial shear box, incorporated with the observations of the soil responses in experiments by others and field responses of in situ soils during large earthquakes. According to a postulate of water pressure transmission in a transient and nearly undrained condition, the soil responses, especially pore water pressure changes, at various locations can quickly affect the soil behavior at different depths and locations. Therefore, it is necessary to consider the seismic response of soils globally within all pertinent soil strata at different depths, not just that of a particular location, in order to fully understand the soil behavior of soils under earthquake loading. Some important implications and their significances are also discussed. Due to the drastic differences of soil behavior before and after liquefaction, the assessment of the state of the soil, liquefied or non-liquefied, is essential in the analysis and design of geotechnical structures.

Key words: saturated sandy soil, water pressure, liquefaction, laboratory tests, earthquake.

1. INTRODUCTION

Since Taiwan is situated in a high seismicity area where large earthquakes occur frequently and cause major damage, the design and construction of geotechnical structures all have to consider the effects induced by large earthquakes. Especially owing to the occurrence of the disastrous Chi-Chi Earthquake on September 21, 1999, a great amount of effort has been spent on the study of seismicity and earthquake engineering in Taiwan. There are also considerable advances in the area of geotechnical earthquake engineering since then.

Prior to the Chi-Chi Earthquake, because insufficient data and analysis of local field information were obtained during the previous earthquakes in Taiwan, the research and design in geotechnical earthquake engineering generally followed the methods developed in the United States and Japan. However, due to the particular geology and soil conditions in Taiwan, it was found that the ground responses and the geotechnical damage during the Chi-Chi Earthquake are quite different from those that occurred in other countries. Therefore, further studies, including site characterization, analyses, and experiments, must be performed to consider the local situations, such as near-fault earthquake loading and soils with high fines contents.

Furthermore, due to the recent earthquakes of much larger magnitudes, the specified design earthquake loading has been raised substantially. As a result, the design methods in the geotechnical engineering would move towards the performance-based design (ISO, 2004) rather than the working stress design. In the performance-based design, the performance of a structure after yield (failure) must satisfy the requirements in the regulations. That is, the behavior of geo-materials and their interaction with the structures, both before and after failure, must be understood to assess the design and performance of these structures and remediation measures.

Not only do earthquakes increase the external loading on the structures, but they also induce the pore water pressure in the saturated soils and reduce the effective stress in the soil mass. Thus, the stiffness and strength of the soil decrease accordingly, and in some cases this may lead to soil liquefaction followed by large deformations or failure of structures resulting in unsatisfactory performance. In geotechnical engineering, because of the inadequacies and difficulties in the theories and experiments, the understanding of the behavior after failure of soil and rock is very poor at this time. This therefore hinders the progress of performance-based design in geotechnical engineering.

Besides the field observations and analyses of the data from the sites where failures or specific phenomena occurred during earthquakes, laboratory experiments, including small soil element tests, centrifuge tests, and large 1 g shaking table tests, are performed to simulate the field conditions for the study of soil behavior under earthquake loading. Based on the experimental data, field observations available to the author, and the pertinent data by other researchers, this paper presents the studies and the inference by the author of the responses of the saturated sandy soils to earthquake loading.

The effective stress in the soil is regarded as the controlling factor on the behavior of saturated sand. As the external total overburden stress generally remains unchanged, the changes in pore water pressure are the main parameter affecting the responses of saturated sand under earthquake loading. Therefore, the discussions in this paper will emphasize on the generation, changes, and transmission of the pore water pressure and their effects on the behavior of saturated sand. Along this line, the liquefaction of soil in this paper is defined as the state of soil

Manuscript received January 7, 2006; revised March 28, 2006; accepted April 1, 2006.
Professor, Department of Civil Engineering, National Taiwan University, Taipei 10617, Taiwan, R.O.C. (e-mail: ueng@ntu.edu.tw).
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with its pore water pressure, u, equal to the total overburden pressure, σ_v, *i.e.*, the effective stress, $\sigma'_v = 0$. In other words, the soil is at a state of liquefaction when the excess pore pressure ratio, $r_u = u_e / \sigma'_v$, reaches 1.0.

2. EXPERIMENTS TO SIMULATE SOIL RESPONSES UNDER EARTHQUAKE LOADING

In order to study the soil behavior, such as strain-stress relationship and liquefaction, small element tests are commonly conducted in the laboratory using triaxial apparatus, simple shear devices, and torsional shear apparatus under regular or irregular dynamic loads. Since the uniformity, saturation and properties (density and fines content), and the stress conditions of the specimen are easier to control in the element tests, they are often used for the study of sand under various given loading conditions and state parameters. In these types of tests, the distributions of stresses and deformations within the soil elements are significantly affected by the specimen boundary conditions, and the loading conditions are generally not the true field situations due to the limitations of the loading devices and the size of specimens. Thus, there are limitations on the validity and the uses of these experiment results.

In the recent years, there are more experiments using larger soil specimens on shaking tables that can reproduce the actual seismic ground shaking in either centrifuge tests or large scale 1 g shaking table tests, *e.g.*, Hushmand *et al*. (1988); Taylor *et al*. (1995). Under well-controlled boundary conditions, these tests can reasonably simulate the field initial stress conditions and the anticipated earthquake loading. Thus, the soil behaviors under a more realistic seismic loading condition can be observed and analyzed. Shaking tests using real size structures inside a large soil mass on a huge shaking table are being attempted to reproduce the field stress conditions and to minimize the scale effect (*e.g.*, Tamura *et al*., 2001; Sato *et al*., 2004; Suzuki *et al*., 2005). Shaking table tests on a large-size specimen have the following advantages:

(1) the loading on the soil can be simulated more closely to the field conditions,
(2) there is less boundary effect on the specimen,
(3) it is easier to install the measuring instrumentation inside the specimen,
(4) the size of instruments is relative small compared to the specimen size and their influence on the soil behavior is minimal, and
(5) a better distribution of the measured values within the soil specimen and their changes with time during shaking tests can be obtained from the instrumentation at various locations.

Other field experiments utilizing blasts, shakers, or falling hammers as the vibration sources were also performed to study the soil behavior under shaking (*e.g.*, Ashford *et al*., 2004; Stokoe *et al*., 2004; Chang and Hsu, 2005). Field instruments were also installed at site to observe the soil responses to the anticipated future earthquakes.

The author has performed 1 g shaking table tests using the biaxial laminar shear box developed by the author at the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. This biaxial laminar shear box at NCREE can be used to simulate a soil stratum of about 1.5 m under a multidirectional shaking on a horizontal plane. Figure 1 shows the large biaxial laminar shear box on the shaking table. The performance and instrumentation in the shear box were tested and verified, and they were found satisfactory (Ueng *et al*., 2006). Nine series of shaking table tests on clean sand in the shear box have been performed at NCREE since August 2002. A large amount of data were obtained from the rather densely placed sensors inside and outside the sand specimen. The observations and analyses of the soil behavior during the shaking table tests using the biaxial laminar shear box provide the primary bases for the following discussions and author's inference on the behavior of saturated sandy soils during earthquakes. It should be noted that for all these shaking table test results and most field observations available to the author, water drainage only occurred through the upper surface of the soil stratum. In reality, the long drainage path through the bottom of the soil stratum should also render very limited bottom drainage under transient seismic loading.

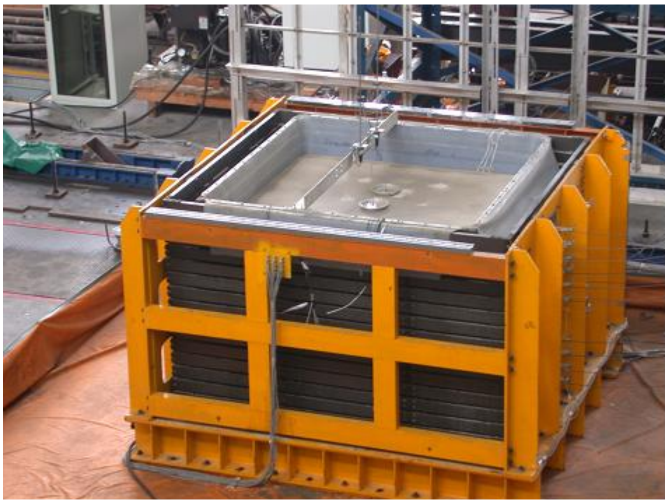


Fig. 1 Large biaxial laminar shear box on shaking table at NCREE

3. PORE WATER PRESSURE CHANGES AND LIQUEFACTION IN SHAKING TABLE TESTS

A fine silica sand from Vietnam was used to prepare the sand specimen for the shaking table tests at NCREE. The representative gradation curve of this sand is shown in Fig. 2. The maximum void ratio, e_{max}, and the minimum void ratio, e_{min}, range from 0.887 to 0.912 and 0.569 to 0.610, respectively, for different batches of sand used in the tests. One- and multi-directional shaking motions including sinusoidal waves (with frequencies from 1 to 8 Hz and amplitudes, A_{max}, from 0.03 to 0.15 g) and real acceleration records, full and reduced amplitudes, from various earthquakes were applied. Miniature piezometers and accelerometers were installed within the sand specimen for pore water pressure and acceleration measurements at different locations and depths in the soil during shaking. Transducers for displacements and accelerations were also placed on different layers of the inner and the outer frames and different locations on the outside walls. Some important observations during the shaking tests are as follows:

3.1 Depth of Liquefaction

For a homogeneous sand stratum, the shallower part is always easier to liquefy than the deeper soil during an earthquake shaking. With an impervious bottom of the shear box, there is an upward hydraulic gradient during and after shaking, and r_u in the deeper soil is usually lower than that in the shallower layer. Figure 3 shows the pore water changes at different depths during a shaking test at NCREE. In Fig. 3, and hereafter in this paper, the pore water pressure is presented in terms of water height (mm). This plot indicates that the sand at a depth of 92 mm liquefied first, followed by the deeper soil, but does not liquefy below the depth of about 898 mm. It also shows that the excess pore water pressure at the shallower depth generally does not exceed and at most equal the pressure at the deeper depth. The same observations were reported by others in their shaking table tests under 1 g or centrifuge condition (*e.g.*, Van Laak *et al.*, 1994). Field observations during earthquakes also showed that liquefaction occurred in the sandy soil at a shallower depth rather than in the deeper soils, although it may be owing to the possible higher density of the deeper soils.

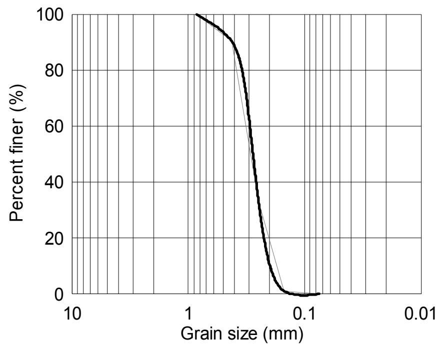


Fig. 2 Grain size distribution of Vietnam sand

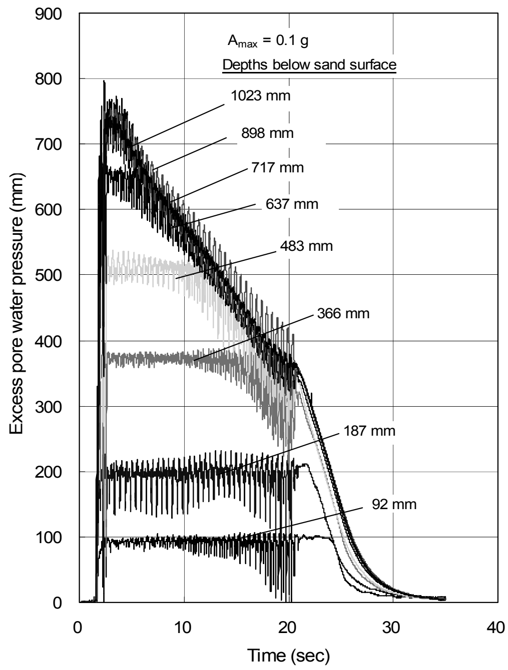


Fig. 3 Pore water changes at various depths during a shaking test, October 2004

For a uniform stratum of sandy soil tested on the shaking table, the above-mentioned observed phenomena seem to contradict the common understanding of the liquefaction mechanism. According to Seed (1979), the equivalent cyclic shear stress ratio (CSR) acting on a soil element in a level ground is:

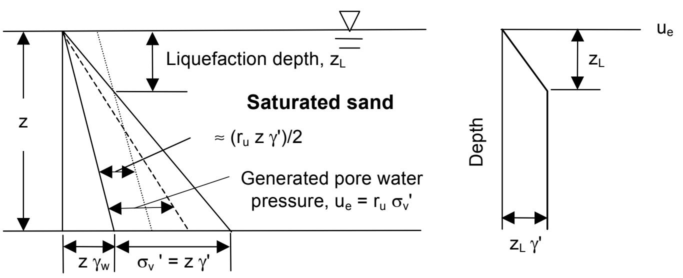
$$\mathrm{CSR} = 0.65 \frac{A_{\max}}{g} \frac{\sigma_v}{\sigma'_v} r_d \tag{1}$$

where $A_{\max}$ = peak ground acceleration of an earthquake, m/s^2;
g = acceleration of gravity = 9.81 m/s^2;
σ_v = total vertical overburden stress, Pa;
σ'_v = effective vertical overburden stress, Pa;
r_d = reduction factor considering the flexibility of soil stratum.

Within the soil depth (≈ 1.5 m) of a shaking table test (or within the field liquefiable strata near the ground surface), r_d should be near 1.0, and the value of σ_v / σ'_v should be constant throughout the whole uniform stratum of saturated soil. Therefore, the soil should undertake about the same shear stress ratio within the whole depth according to Eq. (1). As a result, the excess pore pressure ratio, r_u, induced by the cyclic loading at a given time should be the same at every depth of the specimen (*e.g.*, De Alba *et al.,* 1976). That is, the whole soil stratum should liquefy at the same time if the earthquake loading exceeds the cyclic resistance stress ratio (CRR). Additionally, the deeper soil with a larger effective overburden pressure should exhibit a lower liquefaction resistance (Seed and Harder, 1990), and liquefaction would occur first at the deeper depth rather than at the shallower depth.

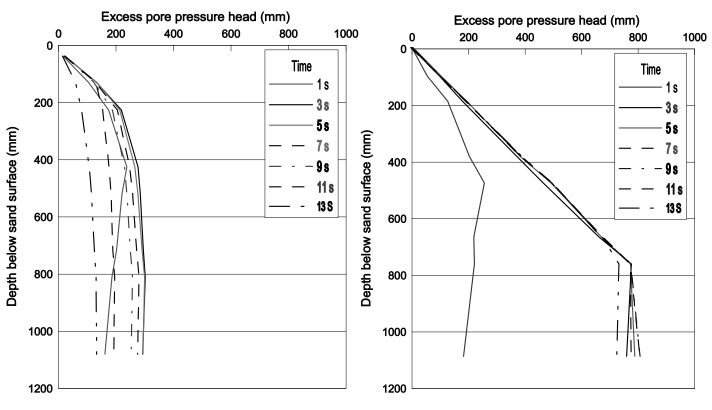
This contradictory behavior of easier liquefaction of the shallower soil is to be explained as follows. Considering a level homogeneous soil stratum, either on the very top of the ground or beneath some layers of other soils, undergoing an earthquake shaking as shown in Fig. 4(a), the induced excess pore water pressure is $u_e = r_u \sigma'_v$ at all depth in the stratum. If the generation of the water pressure is very rapidly under a dynamic loading, it is usually considered that the soil is under an undrained condition, namely, the pore water is in an enclosed container. This leads to, according to Pascal's principle, the change of pressure being transmitted to all parts of the water within the enclosed container or the soil stratum. (The conditions here are not exactly the same as prescribed by Pascal's principle, but it is easier to understand this phenomenon using the more familiar Pascal's principle.) Furthermore, the transmitting of the pressure change should be immediate if the water and soil particles are assumed incompressible. Therefore, as shown in Fig. 4(a), there will be an increase of pore water pressure of a value of about $(r_u z\ \gamma')/2$ in the whole stratum of soil, where z is the depth of the soil stratum and γ' is the submerged unit weight. The effective stress will be less than 0, or liquefaction occurs in the soil above the depth z_L where the effective overburden pressure, $z_L \gamma'$, is less than the induced pore water pressure, $(r_u z\ \gamma')/2$. As the water pressure in the soil above z_L is greater than the total overburden stress, there should be an uplift, or a volume increase of the soil layer above z_L. The pore water pressure thus reduces to a maximum of the total overburden pressure with a tiny amount of expansion of the soil due to the very high bulk modulus of water. Once this happens, the mechanism of water pressure transmission becomes more complex, and the amount of transmitted water pressure and the pressure distribution at various depths change accordingly. The above argument leads to the distribution of excess pore water pressure as shown in Fig. 4(b) that we see in the shaking table tests. For example, Figure 5 is the excess water pressure distribution along the depth of the sand specimen in a shaking table test at NCREE. The one-directional shaking caused a liquefaction depth of around 220 mm, while the multidirectional shaking induced liquefaction at a deeper depth of about 760 mm with a higher pore water pressure.

It should be specially noted that the above-mentioned water pressure transmission and liquefaction can occur without any water flow or upward hydraulic gradient; it is independent of the type of soil; and it can occur during and/or after earthquake shakings. That is, no water seepage is necessary to induce liquefaction or flow failure. This differs from the proposed flow failure due to seepage by Sento *et al*. (2004). In realty, the water and soil grains are not perfectly incompressible, and there is always drainage at the surface of an in situ sandy soil layer unless it is covered by an impervious, non-liquefiable soil layer. Consequently, there would be some diminished amount and time delay of the pore water pressure transmission depending on the easiness of the boundary drainage, the permeability of the soil, and the compressibility of water and soil particles. Theoretically, there should be water pressure wave propagation but the pressure should reach an equilibrium value in a very short time due to the high damping effect of water movement inside the small pore sizes of the soil. Detailed analyses of the phenomena of water pressure generation and transmission are outside the scope of this paper.



(a) Pore water pressure generation (b) After pressure transmitting

Fig. 4 Schematic pore water pressure distribution and its changes in a soil stratum under earthquake shaking



(a) One-dimensional shaking (b) Multidirectional shaking

Fig. 5 Pore water pressure distribution in the sand specimen in a shaking table test (A_{max} = 0.075 g) at NCREE

The above postulate of pore water pressure change and pressure transmission lead to that the pore water pressure change at a given location within a soil stratum does not only depend on the characteristics of the pore pressure generation of the soil at that location but is also affected by the pore water pressure changes of soils at other locations. Thus, the behavior of the soil at one point in the ground would be affected by the soils at other locations within the strata of the pertinent area and depth globally. For example, the results of shaking tests in the VELACS (Hushmand *et al*., 1994) showed that the water pressure changes in the loose and dense sand columns side by side are essentially the same during shaking, even though the denser sand should have a lower pore water pressure generation under the same shaking.

3.2 Significant Implications

Even though there are still studies needed for some of the phenomena observed in the laboratory experiments and field responses of soil during earthquakes, the following significant implications from the above postulate of water pressure transmission can be drawn:

(1) For a soil of high liquefaction-resistance, such as gravelly soil, even though the earthquake shaking may not be strong enough to cause liquefaction of this soil element, the soil can liquefy because its pore water pressure can exceed the total overburden pressure when a water pressure is transmitted from another location, where a high pore water pressure is generated by the earthquake loading. A very likely case is a gravelly soil beneath a shallow impervious soil layer but overlying a loose sand stratum. The liquefaction of the gravelly soil occurs due to the pore pressure generated in the loose sand layer rather than that in the gravelly soil itself under shaking. Many field cases of liquefaction of strong gravelly soils could have occurred due to this mechanism.

(2) The rates of pore water pressure generation and drainage play the essential roles in the water pressure transmission in the soil stratum. A faster water pressure increase but slower water drainage causes the more rapid and less diminished pressure transmission. For a given shaking amplitude, the higher the frequency, the faster the pore water pressure is generated, resulting in a faster water pressure transmitting to the shallower depth. This explains why soil liquefies easier under vibrations of a higher frequency in the shaking table tests. Once the boundary drainage occurs, the water pressure transmission involves seepage of water through the soil. As a result, the hydraulic conductivity of the soil will affect the water pressure transmission, and soils, such as silts, with a lower permeability would delay the pressure increase and liquefaction.

(3) A liquefiable sandy soil layer overlain by an impervious crust, which prevents the drainage and dissipation of the induced water pressure, should have a good chance of liquefaction and possibly a water film beneath the impervious layer if sand boiling does not occur and the liquefied sand particles starts to sediment (Kokusho, 1999; Brennan and Madabhushi, 2005). The quickly transmitted pore pressures and prolonged high water pressures reported by Youd (1999) indicated a poor field drainage condition at the Wildlife site.

(4) At the upper part of a sand stratum, if the effective overburden pressure is low (zero when the stratum is on the surface), it can liquefy readily at the shallower depth. On the other

hand, if this liquefiable soil is overlain by a thick non-liquefiable soil such that the induced pore water pressure at the top of this stratum after aforementioned pressure transmission is still less than the effective overburden pressure, then no liquefaction will occur. This may be one of the reasons why a thicker overlying non-liquefiable causes less liquefaction damage.

(5) If the geological and topographical conditions restrain the uplift of the overlying impervious layer, the pore water pressure in the underneath liquefying soil can possibly exceed the overburden pressure and hydraulic fracturing and/or sand boiling will occur as commonly observed at the historical liquefaction sites (Obermeier, 1996). A record of r_u exceeding 1.0 underneath a 2.9-m silt layer has been reported at the Wildlife site (Youd, 1999).

(6) For a sloping ground surface, the mechanism of water pressure transmission can cause flow failure and lateral spreading due to reduction of effective stress and softening of soil under a sustained shear stress. This shear stress could be substantially less than the residual strength obtained from the laboratory tests without considering the water pressure transmission (Finn, 2000). As mentioned before, this phenomenon does not necessarily require water flow or hydraulic gradient in the soil.

(7) The common methods of liquefaction potential assessment by simply comparing the CRR of a soil element with the CSR induced by an earthquake shaking at that location (*e.g.*, Fig. 6) may not be sufficient without considering the water pressure transmission phenomenon as discussed in this paper. The analysis, analytical or numerical, should be able to take into account the effect of pressure transmission and drainage of pore water during and after earthquake shaking. The discrete element analysis with a microscopic approach by Zeghal and El Shamy (2004) is an example.

(8) The remediation measures against liquefaction and flow failure should also consider the mechanism of water pressure transmission. Short gravel drainage piles in the shallower depths of the liquefiable soil could be as effective as deep stone columns. Further studies are needed on this matter.

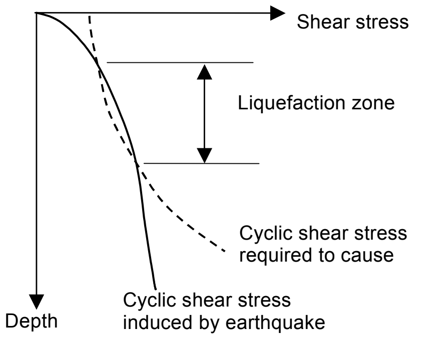


Fig. 6 Evaluation of zone of liquefaction by comparing CSR with CRR

4. SOIL BEHAVIOR BEFORE AND AFTER LIQUEFACTION

Based on the observations in the laboratory and in the field, the behaviors of the soil are drastically different before and after the occurrence of liquefaction. When the clean sand reaches the state of liquefaction ($u_e = \sigma'_v$), it loses its stiffness rapidly and collapses under loading. Due to the loss of stiffness of sand after liquefaction and the hindering of shaking propagation through the softened soil, the shear strain of the liquefied soil in a level ground is usually not as large as that can be produced in the laboratory tests, whereas a larger shear strain can develop if lateral spreading occurred in a sloping ground.

According to the laboratory tests, *e.g.*, Ueng *et al.* (2002), the stiffness of a soil at liquefaction is almost zero, but the soil gains stiffness and strength when the shear strain increases as shown in Fig. 7. Weaver *et al.* (2005) also reported that the lateral resistance of a 0.6 m drilled shaft in liquefied sand induced by blasting is essentially zero until the pile displacement reached approximately 50 mm. The resistance of the liquefied soil increased with displacement as great as 150 mm. The resistance could even exceed the ultimate static resistance. This phenomenon is an important consideration in the performance-based design.

The most important and fundamental difference in sand before and after liquefaction is the contact condition between grains. Prior to liquefaction, there are contacts between most grains to provide an "effective stress" in the soil. While liquefaction occurs at a zero effective stress, grains lose their contacts. Thus, the soil becomes a mixture of water and separated soil particles. The state of the soil can be seen as a fluid rather than a solid. After the external shaking or other agitations diminish, the soil particles start to sediment and the contacts (and effective stress) resume from the bottom upwards. In another situation, when the liquefied soil undergoes a shear strain, the grains start to make contacts and the soil regains its stiffness and strength when the number of contacts increases. The dilatancy during the grain contact process also results in the pore pressure reduction and the effective stress increase. Eventually, the sand-water mixture returns to the state of a solid. This phenomenon is similar to a "phase change" from solid phase to liquid phase and from liquid phase to solid phase. It should be noted the "phase change" here is different from "phase transformation," which is the transformation of the volume change tendency from contraction to expansion. Figure 8 shows an example that the sand specimen above ≈ 396 mm liquefied, became non-liquefied, and re-liquefied again during a shaking table test at NCREE. Kazama *et al.* (2003) also reported the similar phenomenon in their centrifuge liquefaction tests on gravelly decomposed granite soil. The mechanism of this phenomenon is not fully understood yet. Because of the drastic differences between the properties of liquefied

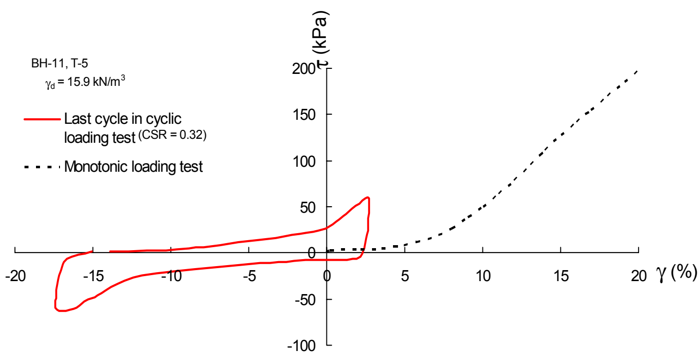


Fig. 7 Stress-strain relation of liquefied Yuan-Lin sand

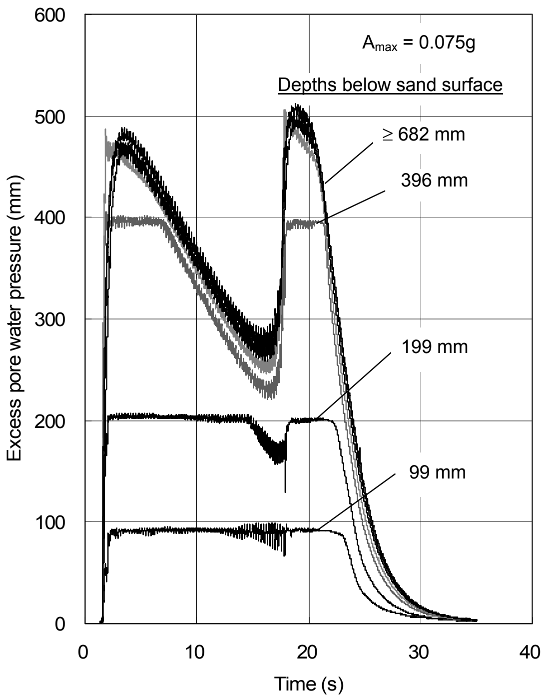
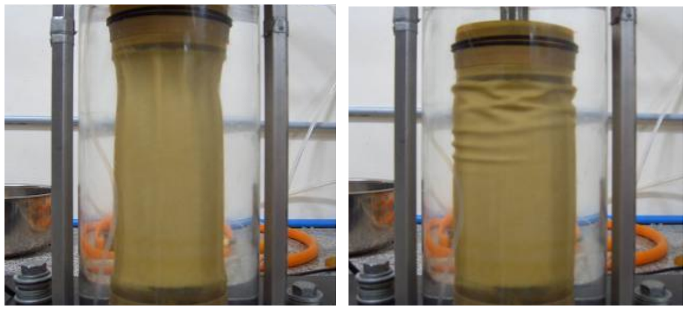


Fig. 8 Pore water pressure changes indicating liquefaction and non-liquefaction of sand at various depths during a shaking test, October 2004



(a) extension (b) compression

Fig. 9 Necking of specimen in a cyclic triaxial test

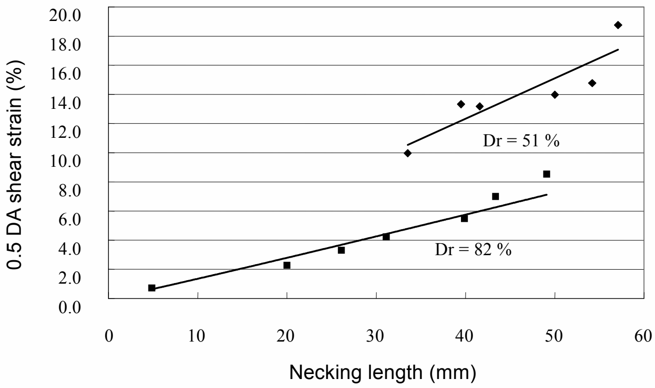


Fig. 10 Variations of maximum shear strain versus necking length of a Vietnam sand specimen (Shi, 2005)

and non-liquefied sand, it is very important to distinguish the state of soil, whether liquefied or not, and the changes of the state with time in the soil-structure-interaction analysis.

Once the soil has reached the state of liquefaction, even though there is an upward hydraulic gradient for water within the layer of the liquefied soil, the water flow is no longer like the ordinary seepage situation. It is the sedimentation of soil particles rather than water flow through the soil pore space. That is, the volume change of the liquefied soil results from the rearrangement of soil particles while the volume change before liquefaction is induced by the increase of effective stress. Accordingly, different settlement behaviors of sand before and after liquefaction were observed during shaking table tests as will be discussed later. This is similar to the void redistribution phenomenon discussed by Boulanger (1999) but with a different interpretation.

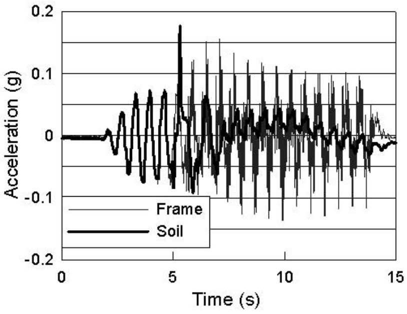
4.1 Post-Liquefaction Behavior Based on Laboratory Tests on Soil Element

Many researchers have attempted to study the behavior of soils after liquefaction using laboratory element tests such as cyclic triaxial tests, cyclic simple shear tests, and cyclic torsional shear tests (Stark *et al*., 1998). Besides many affecting factors on these tests given by Vaid and Sivathayalan (1999), one of the most difficult problems is the non-uniformity of soil conditions within the specimen. The stress, strain, density, water content, pore water pressure and volume change vary drastically within a small soil element when it reaches liquefaction. It is uncertain about the meaning and representation of these measured quantities after liquefaction.

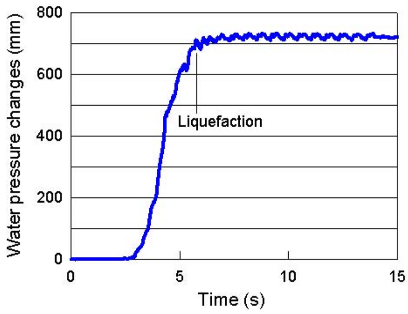
Figure 9 is the commonly observed necking of specimen after liquefaction in the cyclic triaxial test. In the zone of necking, the soil is very much different from the other part of the specimen. The soil density is very low, and there is no effective stress and very little resistance (except the membrane) to the applied loading. Obviously, the applied stresses and measured strains of the specimen are no longer the true stress and strain in the soil element any more. Figure 10 shows that the measured maximum shear strains using external LVDT are proportional to the necking lengths determined using photo image taken during the tests (Shi, 2005). Apparently, the measured strain reflects the change of the length of the necking zone rather than the strain of the whole soil element that is non-uniform after liquefaction. Therefore, the test results related to the measured shear strain of the soil element after liquefaction are probably not very meaningful.

4.2 Measurements of Post-Liquefaction Responses of Large Soil Specimen

In the shaking table tests at NCREE, accelerations measured by the accelerometers within the soil and those on the frames during the shaking tests are compared. The acceleration time histories within the soil and those of the inner frame at the same depth (≈ 0.80 m below the sand surface) and pore water pressure changes near the accelerometer within the soil during a Y-direction sinusoidal shaking with the amplitudes of A_{max} = 0.075g are shown in Fig. 11. It depicts that the induced acceleration within the soil and that of the frame were the same before liquefaction, whereas the accelerations measured within the soil decreased substantially after liquefaction while those on the frame became irregular with spikes. Such behavior might be due to the sudden loss of stiffness of the soil when liquefaction occurred. At this moment, the sensors were possibly decoupled

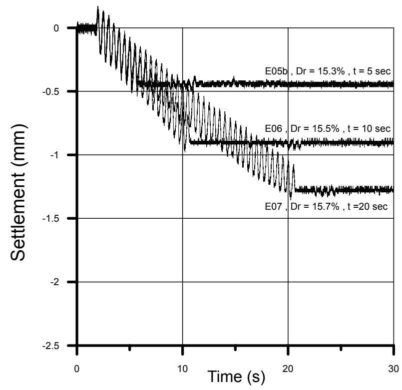


(a) Acceleration time history

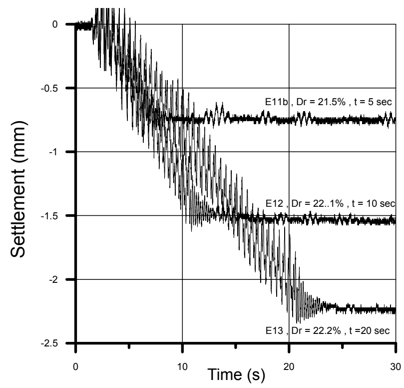


(b) Water pressure changes

Fig. 11 Measured acceleration and pore pressure changes at 0.80 m below sand surface under a shaking of A_{max} = 0.075 g, January 2003

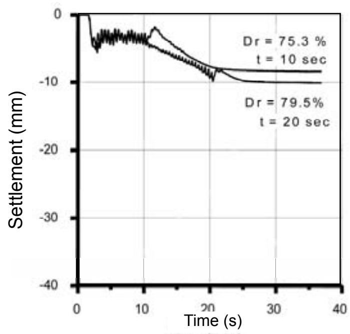


(a) 1-D shaking

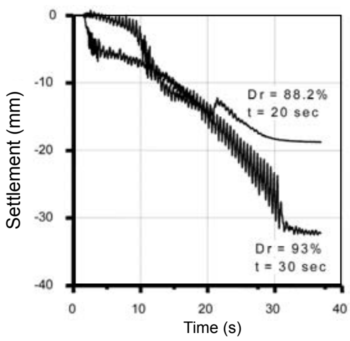


(b) 2-D shaking

Fig. 12 Settlements without liquefaction during shaking tests, April 2004



(a) 1-D shaking



(b) 2-D shaking

Fig. 13 Settlements with liquefaction during shaking tests, April 2004

from the fluid-like soil and the orientations of the sensors probably deviate from the original directions. Thus, the measured movements from the sensors within the soil no longer represented the real movements of the liquefied soil. These results indicate that we can use the measured accelerations either within or on the frames of the laminar box to analyze the induced motions during the shaking tests prior to liquefaction, but it is doubtful that the measured movements, both magnitude and direction, can represent the real motions of the soil after liquefaction. This caution of use of the measured data in the liquefied soil should also extend to the field measurements of the responses of in situ soil during earthquake.

4.3 Settlements and Volumetric Strains of Liquefied Sand

During the shaking table tests on Vietnam sand conducted at NCREE, the settlements of the sand surface were measured during shaking tests using two settlement plates. The heights of 16 locations on the sand surface were also measured manually after each shaking so that the average settlement of the sand specimen can be calculated. It was found that the settlement of the sand specimen is very small (less than ≈ 2 mm) if there was not liquefaction. Significant settlements and volumetric strains were induced only when liquefaction occurred. This is consistent with the very small hydraulic gradient, *i.e.*, little pore water flows out to make room for volume change below the liquefied depth. The settlements of the sand surface resulted from the multidirectional shaking were larger than those under one-directional shaking in both cases of liquefaction and non-liquefaction of the soil. Figure 12 shows the settlements of the sand surface measured by the settlement plates during shaking tests without liquefaction. It can be seen that the multidirectional shakings caused more settlements, which increased linearly with the duration of shaking.

The settlement of the sand surface became significantly large when there is liquefaction. Figure 13 is the surface settlement measured by the settlement plates during shaking with liquefaction. The measured settlements show an erratic fashion because the soil beneath the plates lost it strength and stiffness. It also shows that the settlement continued for a period of time after the end of shaking, whereas, without liquefaction, there is essentially no settlement right after the shaking stopped. This is different from the settlement behavior of non-liquefied soil which shows essentially no settlement after the shaking stopped. In many shaking tests, liquefaction only occurred in the soil of a shallower depth but not the whole sand specimen. The depth of the liquefied sand is determined based on the measurements of miniature piezometers and accelerometers on the inner frames. The soil is considered liquefied when the excess pore water pressure reached a plateau of r_u = 1.0 or the acceleration of the frames became irregular with spikes as mentioned in the previous section (Fig. 11). With consideration of thickness of the liquefied sand, the test results show that the volumetric strain after liquefaction, under sinusoidal shakings of duration of 10 sec, decreases with the relative density of the sand regardless of the amplitude, frequency and directions of shaking as shown in Fig. 14. The relation between volumetric strain of liquefied soil and relative density of sand under sinusoidal shakings of a duration of 10 sec in this study compared with those obtained by Tokimatsu and Seed (1987) are also given in Fig. 14. According to the measured displacements of frames at various depths, the shear strains of the specimen at different depths can be calculated and the maximum shear strain during shaking table tests was found between 0.5% and 3.5%. It indicates that the general trends of volumetric strain changes are similar to those obtained by Tokimatsu and Seed (1987). The results also show a good comparison with the field data given by Tokimatsu and Seed (1987).

It was also found from the shaking table test results that the volumetric strain after liquefaction increases with the shaking duration probably owing to the re-agitation of the settling and settled sand particles. It implies that the duration and/or the magnitude of an earthquake may also affect the settlement of a liquefiable soil.

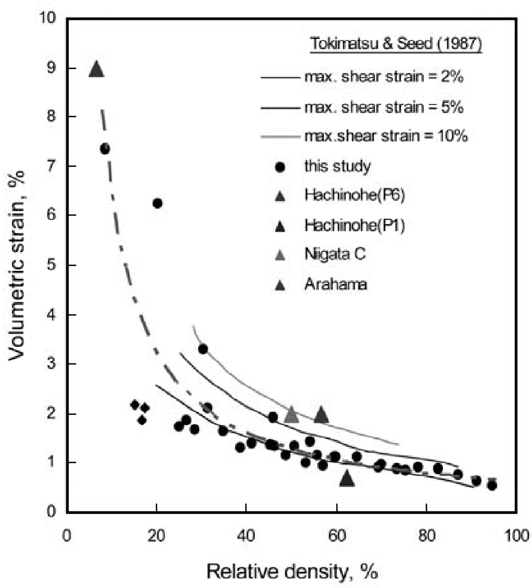


Fig. 14 Volumetric strain of sand after liquefaction versus relative density

5. CONCLUSIONS

During earthquakes, the pore water pressure in a sandy soil will usually increase due to the volume contraction tendency of the soil skeleton. When the pore water pressure exceeds the total overburden pressure and the effective stress = 0, the soil liquefies with no resistance and stiffness. Based on the observations in the laboratory tests on small soil elements, centrifugal shaking table tests, and 1 g shaking table tests on large soil specimens, it is concluded that because of the phenomenon of water pressure transmission during and after an earthquake shaking, the pore water pressure changes in the soil under earthquake loading not only change the soil behavior at that location, but also affect the responses of soil within the whole relevant stratum globally. The easier occurrence of liquefaction of soil at a shallower depth in a uniform soil stratum is an example. The effect of the water pressure transmission depends on the density, rate of pore water pressure generation, boundary drainage, permeability of the soil, *etc*. A soil with a high liquefaction resistance, such as a gravelly soil, can thus liquefy even without a sufficiently strong shaking, but due to a high water pressure transmitted from other soil strata. Therefore, the assessment, analysis, and remediation measures of a liquefiable soil stratum should take into account the phenomenon of water pressure transmission in the soil.

There are substantial differences in the soil behavior prior to, during, and after liquefaction. The soil is more or less like a solid (effective stress > 0) before liquefaction while it becomes a liquid-like material (effective stress = 0) after liquefaction. It is possible that the state of soil can change from solid phase to liquid phase and vice versa during the duration of an earthquake shaking. It is therefore very important to define the zones of liquefaction in the soil-structure-interaction analysis and its changes under the interaction between structures and soil. Development of suitable experiments and measuring systems to obtain meaningful data are critically needed for the study of liquefied soils.

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