Center for Transportation Infrastructure and Safety

Post-Cyclic Behavior of Low Plasticity Silt Under Full And Limited Liquefaction Using Triaxial Compression Testing

by

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at Missouri University of Science and Technology**

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16. Abstract

This project is aimed at graduate research training of students interested in pursuing careers in transportation areas. Each year, financial support was provided to recruit eight new graduate students interested in pursuing a doctoral degree in a transportation area. These students could pursue doctoral studies in any department at Missouri S&T. In departments where a master's degree is the highest degree awarded, students pursuing a master's degree with a thesis option will be considered. Areas stated in the goals, interests and objectives of the State Departments of Transportation and Missouri Department of Transportation in particular were considered for support in this project.

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Research Proposal

1 Introduction

It has been well-known for about thirty years that liquefaction could happen in the silt ground. Lots of liquefaction criteria have been presented to evaluate the susceptibility of the low plasticity silt (Wang, 1979 & 1981; Seed and Idriss, 1983; Koester, 1992; Andrews and Martin, 2000; Seed et al., 2003; Bray et al., 2004b & 2006; Boulanger and Idriss, 2004 & 2006). Loss or damage of property didn't only appear during earthquake. Some of dams or slopes failed not only because of the cyclic loading during earthquake, but also due to the reduced shear strength after earthquake. Most of failures of earth dams have occurred either a few hours or up to 24 hours after an earthquake (Soroush and Soltani-Jigheh, 2009). This phenomenon, called delayed failure or delayed response, proved the need to study the post liquefaction characteristics of soils. Therefore, a research topic in the earthquake engineering field is how to evaluate the post-liquefaction behavior.

There has been some work to study the post-liquefaction behavior of sand (Chern and Lin, 1994; Vaid and Thomas, 1995; Porcino and Caridi 2007; Amini and Trandafir, 2008; Alba and Ballestero, 2008; Ashour et al., 2009). Especially, a National Science Foundation workshop was held at April, 1997 to discuss the post-liquefaction shear strength of granular soils and a report about that was presented (Stark et al., 1997). A general requirement stated in the Byrne and Beaty's keynote paper is that direct tests to determine post-liquefaction strength are generally carried out under consolidated undrained conditions. For sand, this requirement may be reasonable due to the high permeability of sand. However, for the low plasticity silt, the permeability is lower than that of sand. Additionally, the reconsolidation rate depends on the drainage condition in field. If very low permeability layers exist above and below in the liquefied zone, the reconsolidation will take a very long time to finish. A lesson from 2000 Tottoriken-Seibu Earthquake showed that the high pore water pressure could last a long time after the earthquake. A sand boil happened in the Takenouchi Industrial Park on a reclaimed island. The sand boiling continued for 7.5 hours, which was much longer than former experiences in sandy deposit in Niigata. The ground basically consists of nonplastic silt (Towhata, 2008). The liquefied ground should suffer to loading before reconsolidation finishes, especially for dams. Therefore, it is necessary to study the post-liquefaction behavior of the low plasticity sit at different levels of reconsolidation.

During the earthquake, the liquefaction doesn't happen all the time. It depends on the duration and magnitude of earthquake and the properties (with relationship to resistance of liquefaction) of the low plasticity silt. Under the low duration or magnitude of earthquake, the liquefaction doesn't appear in the ground. However, the properties of soil in the ground will be affected. Typically, its shear strength and stiffness will be reduced. The effect of limited cycle of dynamic loading on the soil behavior is called limited liquefaction in the proposal. The post-cyclic behavior of the low plasticity silt under the different levels of limited liquefaction will be studied. Additionally, the effect of plasticity index (PI) on the post-cyclic behavior of the silt under (full) liquefaction will also be investigated. Hopefully, a threshold value can be obtained, above or below which the reduction of shear strength and stiffness will be little and even can be ignored. The studied low plasticity silt will be from Mississippi River Valley in the New Madrid Seismic Zone (NMSZ), which is one of the most seismically susceptible areas in United States. All of work will be conducted using laboratory testing.

Therefore, the work in this research project includes three parts: the post-liquefaction behavior at the different levels of reconsolidation; the post-cyclic behavior under no reconsolidation at the different levels of limited liquefaction; and the effect of PI on the post-liquefaction behavior.

2 Research significance in earthquake engineering

Silt liquefaction was a common phenomenon during the earthquake. During the 1989 Loma Prieta earthquake, the boil of soil happened in the silty ground at Moss Landing, which has a liquid limit of 38, a plasticity index of 17, and a < 5um fraction of 24%. Maximum settlement and lateral deformation that occurred around the Moss Landing Marine Laboratory at Moss Landing in California was 35 cm, and 2.1m, respectively (Boulanger et al. 1998). Liquefaction and cyclic failure of silty soils also occurred during the 1994 Northridge Earthquake. The soil was sandy silt or silty clay, of which average percentage of fines (< 75 um) was 58% and clay content (< 2um) was 20%. More events of liquefaction in the silty ground happened in 1976 Tangshan Earthquake, 1999 Kociaeli earthquake, 1987 Chibaken-Tohooki earthquake, 1995 Hyogoken-Nanbu earthquake, etc. (Bray and Sancio, 2006; Hyde et al., 2006). In the 1999 Kociaeli Earthquake, many buildings settled, titled, or totally collapsed due to liquefaction of silt or silty sand. Building settlements of up to 110 cm were observed. Many buildings suffered severe tilting resulting from the loss of bearing capacity of the foundations (Song et al., 2004).

The low plasticity silt is spread widely in the world. In United States, one kind of typical silt – loess occupies the uppermost stratigraphic position over extensive areas of the central United States and is also found in other parts of the United Sates, including Iowa, Nebraska, Tennessee, Mississippi, Illinois, Alaska, Washington, Colorado, etc. The thickness of loess deposits in the United States varies from 3 ft to 100 ft. Usually the thickest deposits occur adjacent to the Missouri and Mississippi Rivers to the leeward side of the prevailing westerly winds (Puri, 1984). The United States is a county with lots of earthquakes. As one of most significant earthquake zones, the New Madrid Seismic Zone will have a magnitude 6.0 or greater earthquake in years 2000-2050 as high as 98% probability. According to the 2007 update to the National Inventory of Dams, there are more than 80,000 dams in the United States. Approximately one third of these pose a "high" or "significant" hazard to life and property if failure occurs (FEMA, 2009). Additionally, about 70 percent of the 600,000 bridges in the U.S. were constructed prior to 1971 with little or no seismic consideration in design (Anderson, et al. 2002). Once an earthquake happens, it is possible that lots of dams or bridges will be damaged due to the liquefaction of the silt ground if no measurements are taken.

The study of post-cyclic behavior of the low plasticity under full and limited liquefaction is very important to ensure the safety of infrastructure after the earthquake. It has been found that the shear strength and stiffness of soil is normally reduced (Porcino and Caridi, 2007; Yasuhara, 2003; Vaid and Thomas, 1995; Ashour et al, 2009; Soroush and Soltani-Jigheh, 2009). Since the drainage of the low plasticity silt is not good compared to the sand, it takes time to dissipate excess pore water pressure. This is why some dams failed several hours after earthquake stopped. Thus, the post-liquefaction behaviors at the different levels of reconsolidation will be investigated. With the lower cycles of loading or lower shear stress induced by earthquake, there is not enough induced pore water pressure to liquefy silt ground. The excess pore water pressure may last several hours after earthquake. It takes time for the reduced shear strength and stiffness to recover. The study post-cyclic behavior of low plasticity silt under limit liquefaction will be helpful to understand the effect of different levels of cyclic loading on the monotonic behavior. Limited work was found to study the effect of PI on the reduction of post-cyclic monotonic strength and stiffness. If the threshold value of the PI can be obtained, below which the reduction of shear strength and stiffness may be ignored, the post-liquefaction for the soil with the PI less than the threshold value is not necessary to be studied.

3 Goals and Objectives

The objective of this work is to investigate the post-cyclic behavior of low plasticity silt under the full and limited liquefaction and the effect of PI on the post-liquefaction behavior. After determining the index properties, cyclic triaxial testing will be carried out to study the liquefaction resistance. All of specimens will be normally consolidated before cyclic loading. Once after the silt specimens liquefy, the excess pore water pressure will be dissipated at different levels and then implement the different effective reconsolidation pressure. The post-liquefaction monotonic shear testing will be conducted for the specimens experiencing the different levels of reconsolidation in order to study the effect of different levels of reconsolidation on post-liquefaction shear strength. For the specimens after limited liquefaction, no reconsolidation will be allowed to study the effect of different levels of duration of cyclic loading on the post-cyclic behaviors of the low plasticity silt.

The variation of post-liquefaction shear strength with reconsolidation time will be investigated first. One liquefied specimen will be used to reconsolidate completely. The remained excess pore water pressure will be recorded along the time so that the time for different levels of reconsolidation (different effective reconsolidation pressures) can be determined. With the different levels of reconsolidation, the silt specimens will be sheared monotonically in the undrained condition. The post-liquefaction shear strength will be obtained for different levels of reconsolidation. Then, the monotonic shear strength will be known at the different time during the reconsolidation and for different drainage conditions.

Due to limited ground motion induced by earthquake, it is possible that the silt does not liquefy. The silt specimens at different numbers of cycle of dynamic loading won't be reconsolidated and be sheared monotonically. The post-cyclic shear strengths under different levels of cyclic loading will be compared.

The PI of the silt will be changed with adding different amount of bentonite. With changing the PI, the effect of PI on reconsolidation properties and post-liquefaction shear strength and stiffness will be investigated. It is expected that a threshold value of PI can be obtained, below or above which the reduction of monotonic shear strength and stiffness due to cyclic loading may be ignored.

4 Test program

4.1 Post-liquefaction

The soil will reconsolidate and the shear strength and stiffness will be recovered due to dissipation of pore water pressure after it liquefies. For sands, the samples should be consolidated to the initial stress state and then subjected to the expected load path, both prior to and after liquefaction (Stark et a., 1997). However, the rate of recovering of shear strength and stiffness of the low plasticity silt depends on the permeability. Different with the sand, the silt is less permeable and takes some time to recover the shear strength and stiffness. Furthermore, the reconsolidation will also be affected by the drainage condition largely. If the low permeability layers exist under or over the liquefiable soil layer, the time for reconsolidation will be long. So the excess pore water pressure induced by cyclic loading is not necessary to dissipate quickly and can stay there for a long time. The post-liquefaction behaviors will be different after the different levels of reconsolidation. Limited work has been conducted to investigate the post-liquefaction of soil, especially for sand or sandy soil, which can be referenced to study the behavior of the low plasticity silt.

Vaid and Thomas (1995) performed triaxial tests for Fraser River sand using water pluviation to reconstitute specimens. It was found that the liquefied sand deformed at virtually zero stiffness over a large range of axial strain (about 20%). With further straining, the sand always responded in a dilative manner under monotonic loading, even thought the initial sand was contractive under static loading. The post-liquefaction response represented continuously stiffening behavior and an approach to any residual strength was not observed (Fig. 1), regardless of density or effective stress conditions prior to cyclic loading, even after a post-liquefaction strain of 32%. The dilation behavior after liquefaction was also found for Bonneville silty-sand by Amini and Trandafir (2008), for sand by Ashour et al. (2009), for silt by Liu et al.(2007). Vaid and Thomas (1995) explained that during the application of monotonic shearing, the liquefied specimen would undergo rearrangement of the grains. With increasing axial strain, the grains will eventually regain contact, subsequently the pore water pressure starts to decrease and a dilative behavior is seen. The above findings overturns the assumption mentioned before that if the sand is contractive under static testing, its steady state (or residual) strength remains unaltered on monotonic loading following liquefaction induced by cyclic loading (Bryne et al., 1992).

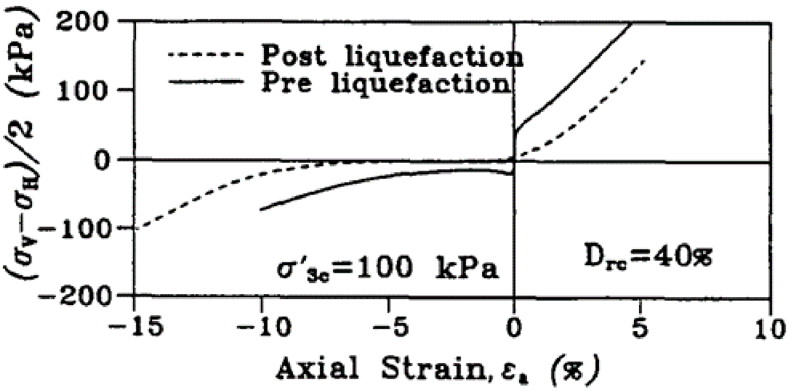


Fig. 1 Comparison of Static and Post-liquefaction Response (Vaid and Thomas, 1995)

Additionally, the effect of some influence factors on the post-liquefaction behaviors was also studied, including density (or unit weight, void ratio), axial strain induced by cyclic loading, and fine content. Vaid and Thomas (1995) found that the increasing rate of stiffness increased as the relative density increases. Liu et al. (2007) found that the threshold strain after which the stiffness increased quickly decreased with increasing the dry unit weight and decreasing maximum double axial strain. This conclusion was also drawn in the Vaid and Thomas's paper (1995). Porcino and Caridi (2007) found that the cyclic resistance of dense sand specimens after being liquefied remained practically unchanged with respect to a new cyclic loading. Conversely, in loose sand specimens a decreased liquefaction resistance could be observed. Such effect was induced by the formation of looser and therefore weaker zones on the top of the specimen after being liquefied. Therefore, the dense sample is more resistant to the reduction of shear strength and more easily recover the stiffness than the loose sample.

The post-liquefaction tests mentioned before were conducted under no reconsolidation condition or under fully reconsolidation condition. There has not been work to study the post-liquefaction behavior under the different levels of reconsolidation. It is interesting to investigate the post-liquefaction behavior under different levels of dissipation of pore water pressure. At the end of liquefaction, the liquefied specimens will be reconsolidated at different levels and then monotonic shear testing will be run in order to compare the initial shear strength and stiffness.

4.2 Limited Liquefaction

Limited liquefaction is a common phenomenon due to the low duration or magnitude of the earthquake or the high liquefaction resistance of ground.

Chern and Lin (1994) carried out cyclic loading and post-cyclic consolidation tests on loose clean sand and silty sand. It was found that the reconsolidation volumetric strain can be related to maximum cyclic strain amplitude or residual pore pressure ratio developed during cyclic loading, regardless of the cyclic stress ratio or the number of stress cycles applied. For the loose sands, with accumulated cyclic single amplitude axial strain less than 1% or residual pore pressure ratio less than 1.0, the magnitude of post-cyclic reconsolidation volumetric strain is relatively small as compared to those developed liquefaction. Similar phenomena were found by Sanin and Wijewickreme (2006) who conducted cyclic direct shear testing for Fraser River Delta silt. Chern and Lin (1994) proposed that the liquefaction condition is a prerequisite to the development of significant amount of volume change due to 1-D reconsolidation of loose level deposit after earthquake shaking (Fig. 2). Ashour et al. (2009) studied the undrained post-cyclic response of sand following limit liquefaction (r_u < 1). The sand under limited liquefaction may experience initial (restrained) contractive behavior that is then followed by dilative behavior. It was presented that the post-cyclic excess water pressure and the associated residual effective confining pressure governed the post-cyclic undrained behavior (stress-strain relationship) of sand (not initial density or confining pressure). Vaid and Thomas (1995) found that the post-cyclic shear behavior become more closer to the initial performance of soil with less excess pore water pressure for limited liquefaction tests (Fig. 3).

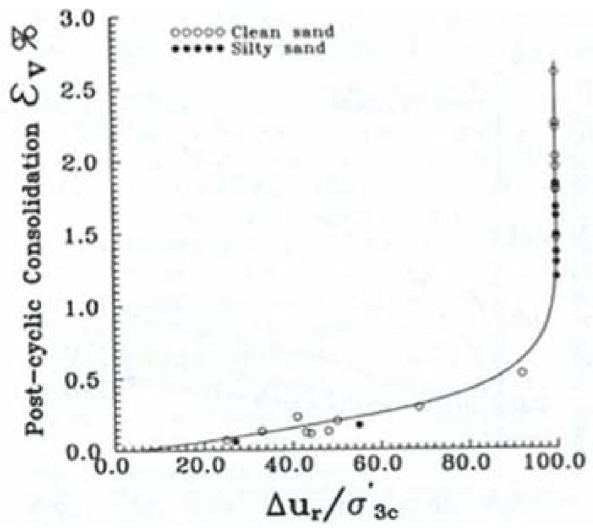


Fig. 2 Relationship Between Residual Pore Pressure and Post-cyclic Consolidation Volumetric strain (Chern and Lin, 1994)

Soroush and Soltani-Jigheh (2009) carried out lots of strain-controlled cyclic triaxial testing for mixed clayey soils (clay-sand and clay gravel mixtures). It was presented that $S_{u(PC)}/S_{u(M)}$ (ratio of post-cyclic undrained shear strength to initial undrained shear strength) and $E_{50(PC)}/E_{50(M)}$ (ratio of secant deformation modulus after post-cyclic test to initial secant deformation modulus) decreased generally as ε_c (cyclic strain) increased, and that the reduction in the deformation modulus was comparatively more pronounced. The specimens' behavior during post-cyclic loading was similar to the behavior of over-consolidated soils. Generally, the value of apparent over-consolidation ratios was proportional to the value of cyclic strains.

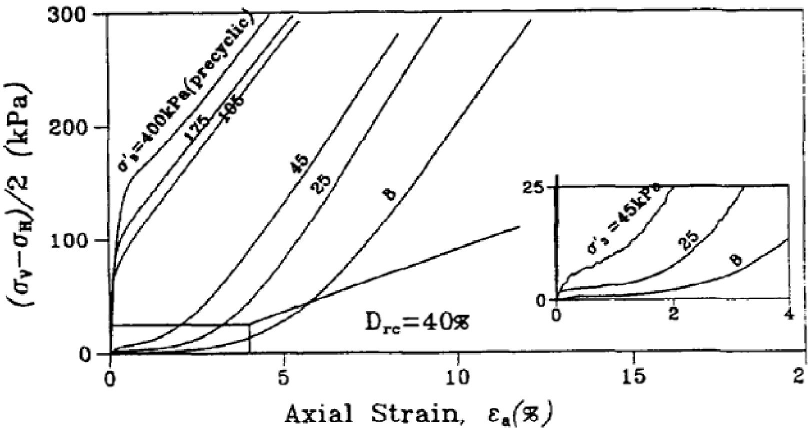


Fig. 3 Post-liquefaction Monotonic Response for $\sigma_3' \neq 0$ Residual States (Vaid and Thomas, (1995)

Besides sand and mixed clayey soil, there are a few work about the post-cyclic behavior of silt. Yasuhara et al. (2003) carried out lots of triaxial tests to study the post-cyclic degradation of strength and stiffness of low plasticity silt. For the same OCR, the post-cyclic shear strength under completely reconsolidation decreased with increasing the pore water pressure ratio at the end of cyclic testing. The stiffness at the beginning of post-cyclic shearing decreased along with the peak deviator stress, with increasing cyclic-induced excess pore pressures. Softening behavior occurred after the strength peaked and this point was reached at increasing strains with increasing OCR. After undrained cyclic loading, the decrease in the initial Young's modulus is more noticeable than that in the undrained strength; and this tendency is more marked for OC specimens. Post-cyclic stiffness of overconsolidated samples can be correlated to the excess pore pressure ratio generated during the undrained cyclic loading. However, compared with normally consolidated samples, the correlation for overconsolidated specimens is not as good. Song (2004) found that slopes for degradation of stiffness ratio $G_{max,cy}/G_{max,NCi}$ of non plasticity silt with increasing cyclic stress ratio increase rapidly at a certain value of the cyclic stress ratio. With increasing initial shear stress (τ_s), the rapid decrement in the stiffness ratio $G_{max,cy}/G_{max,NCI}$ starts at a lower value of the cyclic stress ratio R_{DSS}.

In summary, the volume change of the sand due to the reconsolidation after limited liquefaction is much lower than that after full liquefaction. The excess pore water pressure and effective confining pressure at the end of cyclic loading control the post-cyclic undrained behavior of the sand irrespective of initial density or confining pressure. The reduction of shear strength and stiffness of mixed clay soils are related to the cyclic strain at the end of the cyclic loading. The reduction of the stiffness is more marked than that of shear strength. For low plasticity silt, the excess pore water pressure at the end of cyclic loading governs the reduction of shear strength and stiffness. With more excess water pressure, the reduction will be more.

The reduction of stiffness is more marked than that of shear strength, especially for overconsolidated specimens.

The post-cyclic behavior will be studied for the studied low plasticity silt at different levels of limited liquefaction, which will induce different excess pore water pressure. The post-cyclic monotonic testing will be done under no reconsolidation. There will be different levels of effective reconsolidation stress. The shear strength and stiffness for different levels of limited liquefaction will be compared to those for different levels of reconsolidation with full liquefaction. If the effective consolidation stress governs the post-cyclic monotonic behavior, the shear strength and stiffness will be closed for between the two cases with the same effective consolidation pressure before monotonic shearing.

4.3 Effect of PI on Post-liquefaction Behavior

It has been verified that the plasticity index plays a great role on the liquefaction resistance of silt. The PI may be used as a criterion for estimating the liquefaction potential of soil (Gratchev et al., 2006a, 2006b; Guo and Prakash, 1999). Guo and Prakash (1999) concluded that the liquefaction resistance of undisturbed samples first decreases with an increasing plasticity index up to a PI of about 5, and then increases with an increasing PI.

It can be imagined that the post-liquefaction behavior should be influenced by the PI. However, there are very limited work to study the effect of PI on the reduction of shear strength and stiffness due to the full liquefaction. Song et al. (2004) found that the decreasing tendency of stiffness ratio for non plasticity silt was less marked than that for plastic silt when plotted against the normalized pore pressure. Alba and Ballestero (2008) stated that increasing fines contents was found to decrease the residual strength compared to that clean sand under similar placement conditions. Boulanger (1998) presented that the post-cyclic strength increased with increasing plasticity index.

Therefore, it seems that the reduction of the shear strength and stiffness increase with increasing clay content or PI, based on Song et al. (2004) and Alba and Ballestero (2008). However, these limited cannot yield general conclusion. To study the role of PI on the post-liquefaction behavior for the Mississippi River Alluvial silt, some bentonite will be added to the silt and increase its PI value. With the different PI, the post-liquefaction monotonic testing will be carried out to investigate the variation of reduction of shear strength and stiffness with varying the PI.

5 Work plan

In order to guide the whole work, a general plan was done here. Until now, some parts of them have been done, including index properties measure, specimen preparation improvement, and static triaxial testing.

Task 1 Index Properties

Based on ASTM standard, some tests were carried out to investigate the grain size distribution, Atterberg limits, and specific gravity. Herein, the Atterberg limits were also measured using Fall Cone approach, besides Casagrand approach, since it was very difficult to do Atterberg limit tests for the low plasticity silt using the Casagrand approach. The soil easily cracks when cutting the soil specimen in Casagrand cup and then large error can easily exist. The results from the fall cone approach would be combined with the results from Casagrand approach to determine the Atterberg limits. The liquid limits based on the Fall Cone and Casagrande approaches were 30 and 28, respectively. Koester (1992) studied the relationship between liquid limits determined by Casagrande and Fall Cone approaches. A summary plot is shown in Fig. 4. The liquid limit values measured for the studied Mississippi River Alluvial silt was added in the plot. It can be found the point is located in the scope gotten by lots of people. Therefore, the liquid limits were reasonable. Here, the result from the Casagrande approach was applied. And the plasticity limit was 22, so the plasticity index was 6.

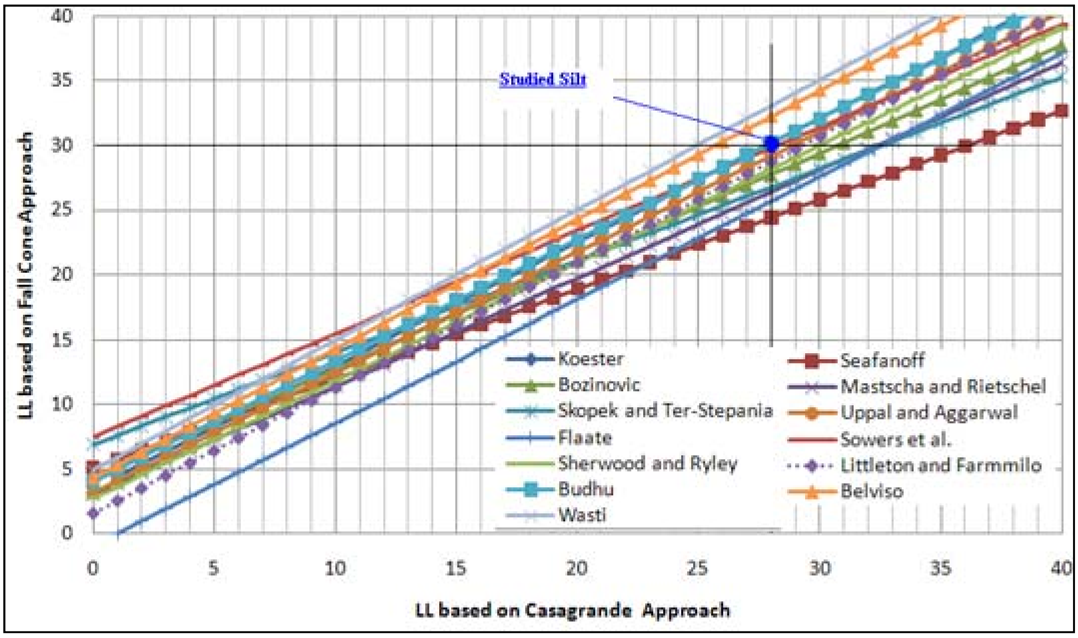


Fig. 4 Relationship between Liquid Limits Determined by Casagrande Approach and Fall Cone Approach

Since the silt is difficultly densified by the vibrating table method, a different method should be used to measure the minimum void ratio for the studied silt. Polito (1999) found that the modified compaction test (ASTM D 1557) gave similar results to the vibrating table test when measuring the minimum void ratio for silty sands. Furthermore, the densification of silt using modified compaction method would be not a problem, so it was carried out to measure minimum void ratio for the studied silt. Since high bulking easily appears for silts when using dry silt to determine void ratios (ASTM D 4254), the maximum void ratio was obtained by allowing slurry to settle out in a graduated cylinder (Bradshaw and Baxter, 2007).

The specific gravity of the studied silt is 2.71 and its minimum and maximum void ratios are 1.60 and 0.44, respectively. Furthermore, the clay content (< 2mm) is 14.5% according to grain size distribution. Based on the criteria by Seed et al. (2003) and Boulanger and Idriss (2004, 2006), the studied low plasticity silt should be susceptible to liquefaction.

Task 2 Specimen Preparation

Izadi (2006, 2008) prepared silt specimen in large scale consolidometer and split mold. A comparison about the different silt specimen approach was make and shown in the following figure. It verifies that the silt specimen prepared in the split mold with top and bottom loading is more uniform and so the split mold will be selected to prepare the silt specimens. Izadi (2008) only used vacuum as the bottom loading to consolidate specimens after the top dead weight, 1.4% difference of water content between top and bottom was yield (Fig. 5). In order to get more uniform specimens, the same vacuum would be put at the top and bottom of the specimens at the same time to consolidate them. The studied silt specimen has 1.2% different of water content along the vertical direction of specimen (Fig. 5). It seems that the shape of water content is more reasonable, since the water content is supposed to be higher at the center due to the longer drainage distance to the top and bottom of the specimen. Therefore, the same vacuum will be put at the top and bottom in the following work after the silt slurry becomes hard under the dead weight and can stand without split mold.

Another problem is that the specimen preparation would be too slow if every specimen was prepared in the same chamber which would be used to do the next procedures, including saturation, consolidation, cyclic loading, and post-cyclic testing. It would take about ten days to finish all of process for one specimen, and so the time of carrying out laboratory testing would be too long. In order to avoid this problem and speed up the process, a tool only for preparing specimen using slurry approach was developed. At the same time, one problem came out. How to move the specimen from the preparation location to the GCTS chamber? This key point is that the specimen should be moved with little disturbance. The figure shows a general idea how to move specimen successfully with very little disturbance. This movement approach yields very little disturbance to the specimen.

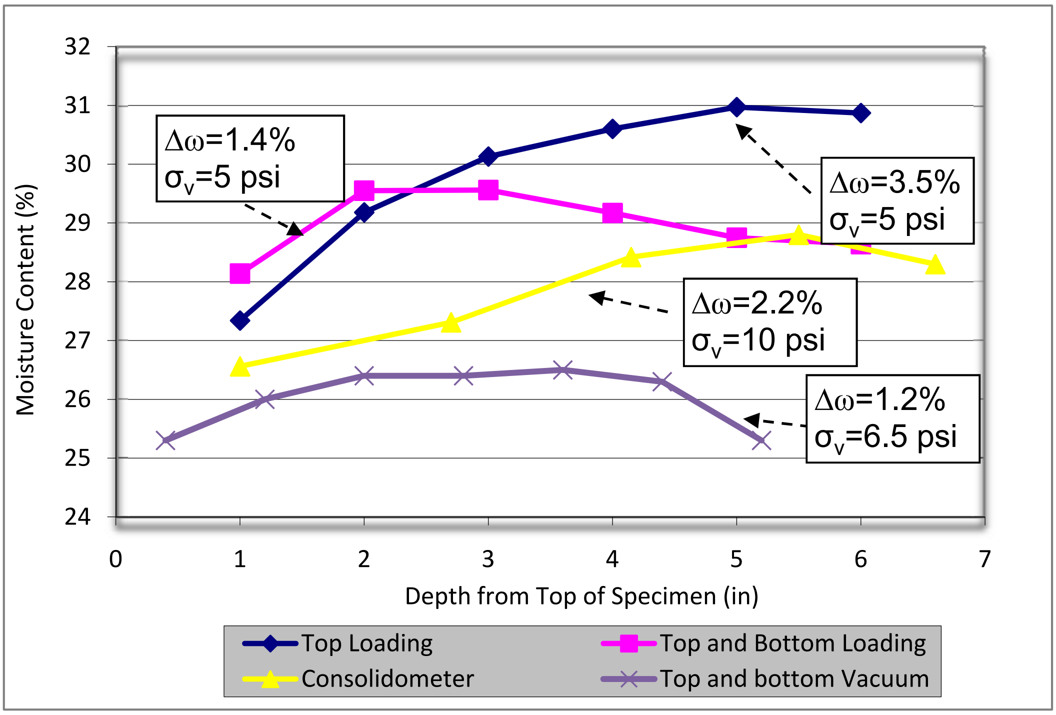


Figure 5 Effect of preparation techniques on the uniformity of specimens

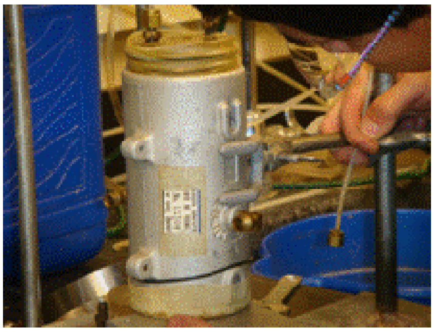
Task 3 Static and Cyclic Triaxial Testing

Low plasticity silts could exhibit dilation behavior even under normally consolidation in the compression (Penman, 1953; Wang, 1982; Fleming and Duncan, 1990; Hoeg. et al., 2000; Silva and Bolton, 2005; Brandon et al., 2006; Boulanger and Idriss, 2006; Izadi, 2006). On the other hand, some people found that the low plasticity silt could also show contraction or plastic stress-strain behavior (Boulanger and Idriss, 2006; Hoeg. et al., 2000). Whether the silt show dilation or contraction behavior depends on OCR, particle shape and specimen preparation approach, besides PI.

To identify the stress-strain behavior of the studied low plasticity silt, several consolidated triaxial tests will be carried out under the different effective consolidation pressure and overconsolidation ratio (OCR = 1, 2, 4, 8). Under every OCR, at least two specimens with different confining pressure would be done for the monotonic compression testing. It is hoped that the most important factor on the monotonic behavior can be obtained. Until now, three tests for the normally consolidated specimens have been finished. All of them showed a little bit dilation. The index properties of the studied are almost the same with the silt used by Izadi (2006). However, the silt showed high dilation behavior in the Izadi's work (Fig. 7). A comparison will be made to find out the reason why the two kinds of materials with the almost same index properties show different stress-strain behavior (dilation or contraction). A Cold Field Emission Gun Scanning Electron Microscope (FEGSEM) will be used to investigate the silt particle shape at the Material Research Center of MST.

(a) Use a different split mold to hold the silt specimen and fix the specimen with a holder

(b) Move away o-rings and stretch up the membrane



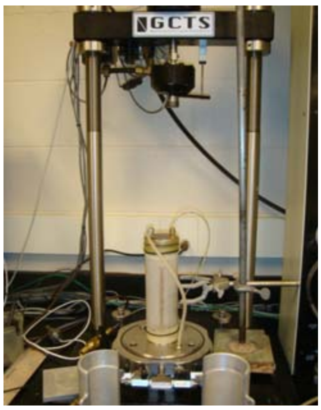
(c) Slide silt specimen to the metal plate

(d) Move the silt specimen to the GCTS platen and fix it with the holder

(e) Stretch down the membrane and o-rings to the GCTS base

(f) Change a filter paper and set the GCTS cap with screw

(g) Put vacuum at the top and bottom of the specimen to remove the air and move away the split mold



(h) The silt specimen are ready for testing

Fig. 6 Movement of Silt Specimen from Preparation Location to GCTS Platen

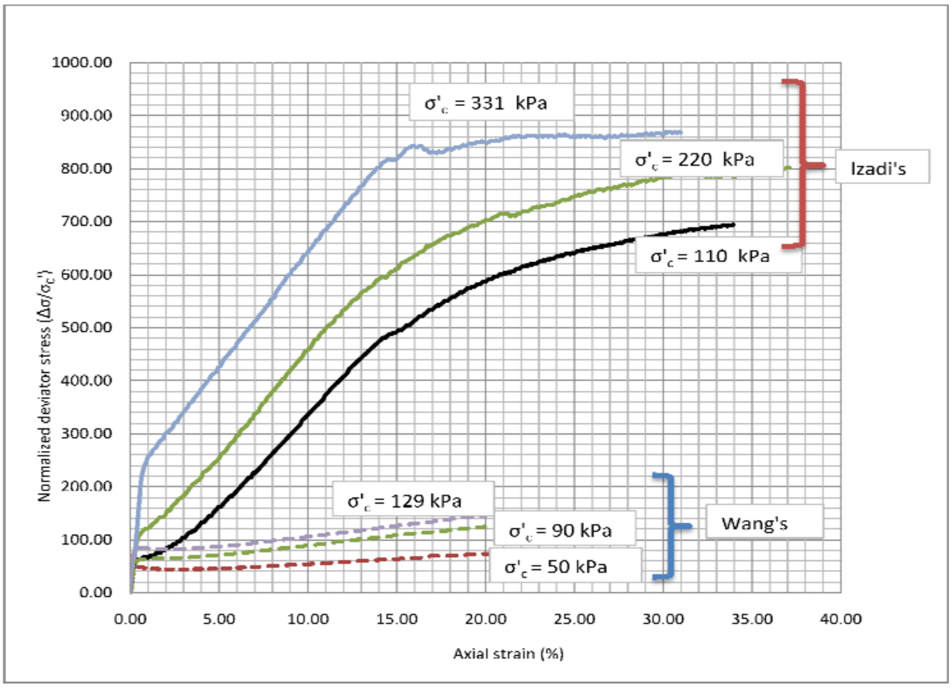


Figure 7 Comparison of Normalized Stress-strain Behavior Between Two Kinds of Silts

The liquefaction potential of the Mississippi River Alluvial silt will be studied to identify whether the silt shows initial liquefaction or cyclic mobility. With the identical OCR and effective consolidation pressure (σ'_c) but different cyclic shear stress, a group of tests consisting of five specimens will be done to plot a curve showing the cyclic shear stress ratio versus the number of cyclic loading based on u = 100% σ'_c (initial liquefaction) or specific axial strain (cyclic mobility).

Task 4 Post – liquefaction

Post-liquefaction monotonic testing will be done under the different reconsolidation levels. Before this, the process of the reconsolidation (i.e. dissipation of excess pore water pressure) will be monitored. Thus, the time at different levels of reconsolidation will be known. After the time for different levels of reconsolidation (0%, 25%, 50%, 75%, 100%) is obtained, the monotonic testing can be carried out under the different levels of the reconsolidation.

To identify the reconsolidation settlement behavior, reconsolidation curve will be plotted to compare the initial consolidation curve (virgin) for the studied silt. Porcino and Caridi (2007) presented that reconsolidation line was parallel to and below the initial consolidation line for their studied sand (Fig. 7). It was verified that the sand became denser but had same consolidation index. There has not been available result showing the reconsolidation behavior.

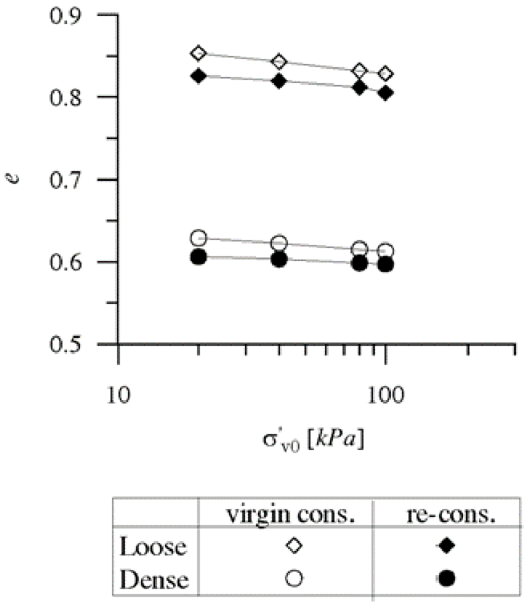


Figure Pre- and Post liquefaction one-dimensional compressibility of Ticino Sand Specimens (Porcino and Caridi, 2007)

It should be noted that the pore water pressure should reach equilibrium before the post-liquefaction monotonic tests. So the pore water pressure will have a little change once closing the drainage valves. After pore water pressure reaches the equilibrium, the monotonic shear tests will be done under the different levels of reconsolidation. The strain-strain behavior (dilation or contraction), shear strength and stiffness will be compared among the specimens with different levels of reconsolidation. Finally, the post-liquefaction properties will be plotted versus reconsolidation time.

Task 5 Pre- liquefaction

The reasons of no liquefaction include low duration and magnitude of earthquake, besides the high liquefaction resistance of ground. The low duration will be applied to induce the limited liquefaction at the same cyclic shear ratio. The number of cycle of loading can be known from the Task 3 and marked as N_{lf}. With the same cyclic stress ratio, the lower numbers of cycle of loading will be set. They will be 0.75 N_{lf}, 0.5 N_{lf}, 0.25 N_{lf}. The soil will have different excess pore water pressure after the different levels of cyclic loading.

With the different excess pore water pressure, the post-cyclic monotonic tests will be done. It is supposed that the post-cyclic monotonic behaviors should be different after the different levels of reconsolidation like the post-liquefaction. Here, the behaviors without any reconsolidation will be investigated for the limited liquefied silt. After the excess pore water reaches the equilibrium in the silt specimen, the monotonic tests will be carried out to get the stress-strain behavior.

It is expected that the effect of different levels of loading or excess pore water pressure induced by cyclic loading on post-cyclic monotonic behavior will be presented. This will be compared to that under the same effective reconsolidation stress but with full liquefaction.

Task 6 Effect of PI on Postliquefaction behavior

Atterberg limits testing should be done first for the silt added with different quantity of bentonite. The variation of PI with different clay content will be presented.

All of specimens with different PI will be tested under the same effective consolidation pressure, cyclic shear stress. After liquefaction happens and reconsolidation finishes completely, the monotonic shear tests will be done to investigate the effect of PI on the reduction of shear strength and stiffness of the studied silt compared to the initial monotonic values. It is expected that a threshold value of PI can be obtained, below or above which the reduction of monotonic shear strength and stiffness due to cyclic loading may be ignored.

3.6 Closing

There has been little work on the behavior of the low plasticity silt, especially its post-cyclic behaviors under full and limited liquefaction. The behavior will be significant for improving or ensuring the safety of infrastructure during and after the earthquake. The low plasticity silt from Mississippi River Alluvial will be identified with several index properties tests and static and liquefaction testing. The influence factors of the monotonic shear strength of the silt will be studied, including the OCR, confining pressure, and void ratio. The liquefaction potential will be investigated with the cyclic triaxial testing. With the basic features of this low plasticity silt, the post-cyclic behavior of low plasticity silt after full and limited liquefaction behaviors will be studied. It is expected that the post-liquefaction behavior will be obtained with different levels of reconsolidation and the recovering of shear strength and stiffness with time will be known. The effect of duration of cyclic loading on the post-cyclic behaviors will be presented via the different levels of limited liquefaction. Finally, the effect of PI on the post-liquefaction behaviors will be investigated with adding different quantity of clay to the silt to obtain a threshold value of PI, below which the reduction of strength or stiffness will be not large.

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