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The Importance of Mineralogy and Grain Compressibility in Understanding Field Behavior of Failures

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THE IMPORTANCE OF MINERALOGY AND GRAIN COMPRESSIBILITY IN UNDERSTANDING FIELD BEHAVIOR OF FAILURES

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ABSTRACT

In this paper, we examine the role of grain mineralogy and compressibility, sample preparation, and shear strain/displacement levels on the shearing behavior of sands using undrained triaxial and constant volume ring shear tests in an attempt to explain some discrepancies observed between field and laboratory behavior. As expected, preparation by moist tamping can produce specimens that are contractive throughout shear, while counterparts prepared using pluviation exhibit dilative behavior at intermediate shear strain/displacement levels (i.e., after initial yield). However, both triaxial and ring shear tests illustrate that some sands consisting of more compressible minerals can exhibit entirely contractive behavior regardless of the sample preparation method. These preliminary tests suggest that laboratory testing of pure quartz sands may result in potentially misleading conclusions regarding the field behavior of mixed mineral soils involved in many liquefaction flow failures and long run-out landslides. Furthermore, grain crushing at larger displacements (larger than those that can be achieved in the triaxial device) results in net contractive response regardless of the sample preparation method or the grain mineralogy. Grain crushing has been observed in shear zones formed during a few well-documented long run-out landslides. The combination of these factors: grain mineralogy and compressibility, particle damage and crushing, and shear zone formation may help to explain some discrepancies observed between field and laboratory behavior of sands.

INTRODUCTION

Liquefaction-induced failures of man-made embankments and natural slopes historically occur preferentially in soils deposited in water by artificial hydraulic filling and in natural fluvial environments. Some investigators have attempted to reproduce the observed field behavior of these failures in the laboratory by performing triaxial tests on pluviated specimens of pure quartz sands, as pluviation is believed to better reproduce in-situ soil structure (e.g., Oda et al., 1978; Miura and Toki, 1984; Ishihara, 1993; Vaid, 1994; Yamamuro and Lade, 1997; Uthayakumar and Vaid, 1998; Yamamuro and Covert, 2001). However, most of these efforts have failed to reproduce flow liquefaction, i.e., the specimens have strain hardened (dilated) during shear and have not been as compressible as the in-situ soil deposits (Ishihara, 1993; Uthayakumar and Vaid, 1998; Yamamuro and Covert, 2001). Similarly, the full-scale CANLEX experiment (Wride and Robertson, 1999) indicated that water deposited in-situ sands are unlikely to be looser than the loosest laboratory specimens achievable by water pluviation.

As a result, many researchers (e.g., Bjerrum et al., 1961; Castro, 1969; Ladd, 1978; Mulilis et al. 1978; Sladen et al., 1985; Kramer and Seed, 1988; Konrad, 1990; Ishihara, 1993; Pitman et al, 1994; Chu and Leong, 2002; Ng et al., 2004; among others) have used moist tamping to reconstitute specimens of clean, quartz sands that are sufficiently loose to strain soften (i.e., contract). In this method, voids are created artificially in the soil structure by developing “apparent cohesion” in the soil resulting from water surface tension between grains, so that upon shearing there is a significant volumetric contraction in the soil. Such specimens can be reconstituted at much higher void ratios than most in-situ sands, as well as sands reconstituted by water or dry pluviation.

Therefore, we anticipate that some factors other than the compressibility of the initial soil structure must be involved in many liquefaction-induced failures of embankments and slopes. Such factors may be related to the compressibility of the soil grains, particle damage and crushing, and shear zone formation. In order to investigate these factors, we present the preliminary results of a set of undrained triaxial compression and constant volume ring shear tests performed on three sandy soils with different mineralogies. We performed the ring shear tests to overcome the well-known displacement limit of the triaxial device and better mimic the displacement magnitudes experienced in many liquefaction-induced failures.

TESTING PROGRAM

We selected three sands for this study: an Illinois River sand (IR), a Mississippi River sand (MR), and Ottawa 20/40 sand (CO). The Illinois River sand is a medium-grained, uniform alluvial sediment from the Illinois River, with a fines content of less than 1% by weight. The particles are rounded to sub-rounded, and consist primarily of quartz with traces of muscovite, chlorite, and hematite (Mueller 2000). The Mississippi River sand is very fine silica sand with sub-angular to sub-rounded particles and contains about 70% albite, 21% quartz, and 5% calcite that we sampled near Cape Girardeau, Missouri. Ottawa 20/40 sand is a commercially-available, medium-grained, uniform, pure quartz sand with round particles from Ottawa, Illinois. Figure 1 presents the average grain size distributions of these sands, and Table 1 presents their physical characteristics.

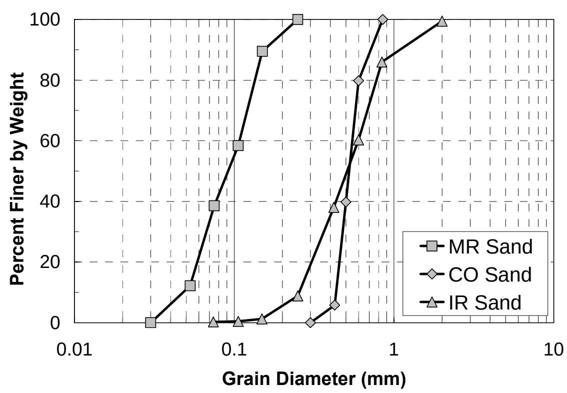

Fig. 1. Particle size distribution of the test sands

Table 1. Physical characteristics of the sands

Sand	Gs[a]	e_{max}[b]	e_{min}[b]
IR	2.63	0.757	0.464
MR	2.65	1.0378	0.563
CO	2.64	0.679	0.391

[a] Defined using the ASTM D854-00 method.
[b] Defined using Yamamuro and Lade, (1997) method.

We prepared all of the specimens in the "loosest possible" condition producible by moist tamping and air pluviation. In the moist tamping method, the sand was moistened and thoroughly mixed with 5% water, and then poured and gently tamped in 20 layers into the specimen container. Under-compaction as proposed by Mulilis et al. (1978) was used to achieve a relatively uniform density throughout the specimen. We employed moist tamping for some specimens because this was the only method that was capable of producing IR and CO specimens loose enough to contract throughout shear. In the air pluviation method, dry sand was poured into a funnel with its tip resting on the bottom of the specimen mold. Then the funnel was gently raised to deposit the particles with nearly zero drop height. This technique produced the loosest possible structure using air pluviation (Lade et al., 1998) and reduced segregation between the fine and coarse grains. We used air pluviation alone to prepare specimens of MR Sand, because moist tamped samples of this sand deformed severely even under very small initial confining pressures. Table 2 provides the details for each test in this study. We selected the consolidation pressures to be similar to consolidation pressures that existed in some larger liquefaction flow failures, e.g., Calaveras Dam (Hazen 1918), Fort Peck Dam (Casagrande 1965), Aberfan Tip (Bishop 1973), and Lower San Fernando Dam (Seed et al. 1973). Table 2 includes the relative densities of the samples after deposition (Dr_i) and after consolidation (Dr_c). As expected, the loosest possible void ratio produced by moist tamping was considerably looser than the loosest possible void ratio produced by air pluviation.

Table 2. Details for each test performed for this study

Test[a]	Sand	Method[b]	σ'_{nc}[c] (kPa)	e_i[d]	Dr_i (%)	e_c[e]	Dr_c (%)
TX1	CO	MT	358	0.851	-60	0.787	-38
TX2	CO	AP	366	0.673	2	0.643	13
TX3	IR	MT	569	0.780	-8	0.659	33
TX4	IR	AP	573	0.691	23	0.627	44
TX5	MR	AP	272	0.816	47	0.645	83
RS1	CO	MT	375	0.622	-2	0.631	17
RS2	CO	AP	357	0.686	20	0.585	33
RS3	IR	MT	266	0.763	-2	0.672	29
RS4	IR	AP	273	0.702	19	0.648	37
RS5	MR	AP	395	0.856	38	0.756	59

[a] TX = Triaxial compression; RS = Ring shear
[b] MT = Moist tamped; AP = Air pluviated
[c] Final normal consolidation pressure.
[d] Void ratio after deposition.
[e] Void ratio after consolidation.

For the undrained triaxial tests, we used lubricated end platens to reduce end restraint. We flushed specimens with carbon dioxide followed by de-aired water, and back-pressured the sample until a pore pressure parameter (B) of at least 0.97 was obtained. After consolidation, the drainage lines were closed and shearing commenced as soon as possible to prevent secondary compression from affecting the void ratio. The specimens were sheared under undrained conditions at a rate of 0.127 cm/min to an axial strain of 25%. To overcome the triaxial device's displacement limitations, we performed several constant volume ring shear tests using the newly developed ring shear apparatus at the University of Illinois. The University of Illinois ring shear device has inner and outer diameters of 20.3cm and 27.0cm, respectively, and a height of 2.6cm. The ratio of the outer to inner ring diameter is 1.33. This diameter ratio results in an error of less than 3% at the peak shear stress due to strain non-uniformity (Hvorslev, 1939). The wide sample section (3.3 cm) also minimizes wall friction effects. In the ring shear tests, each sand specimen was deposited in the ring shaped chamber of the apparatus, consolidated to the target effective stress, and sheared at a rate of 18.6 cm/min in a constant volume condition. Sadrekarimi and Olson (2007a) provide further detail of the ring shear device, specimen preparation, and testing method.

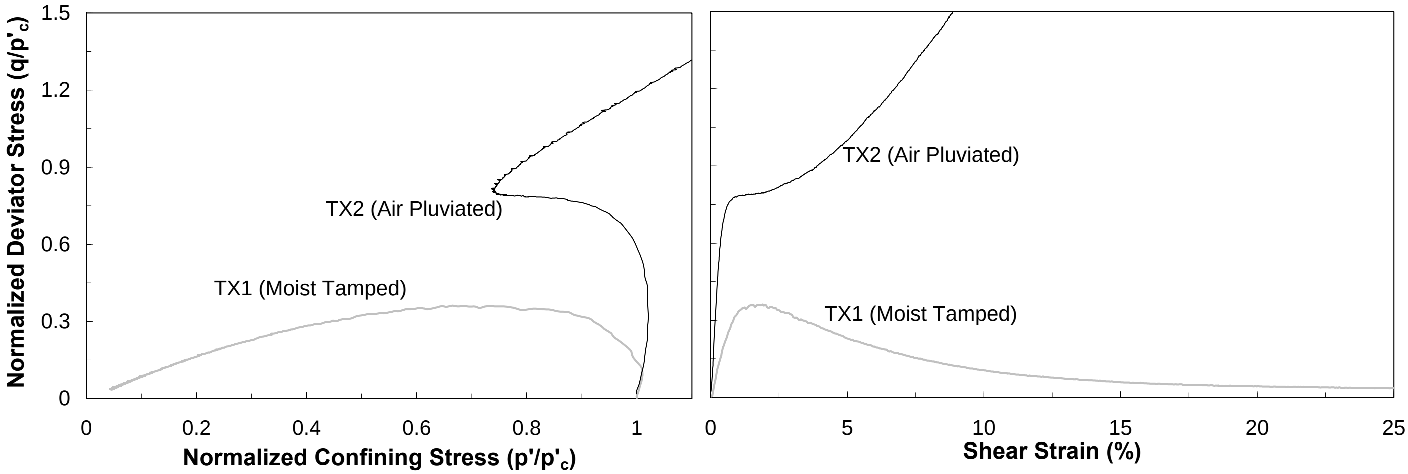

Fig. 2. Normalized stress paths and shear stress – strain plots of triaxial compression tests on specimens of CO sand

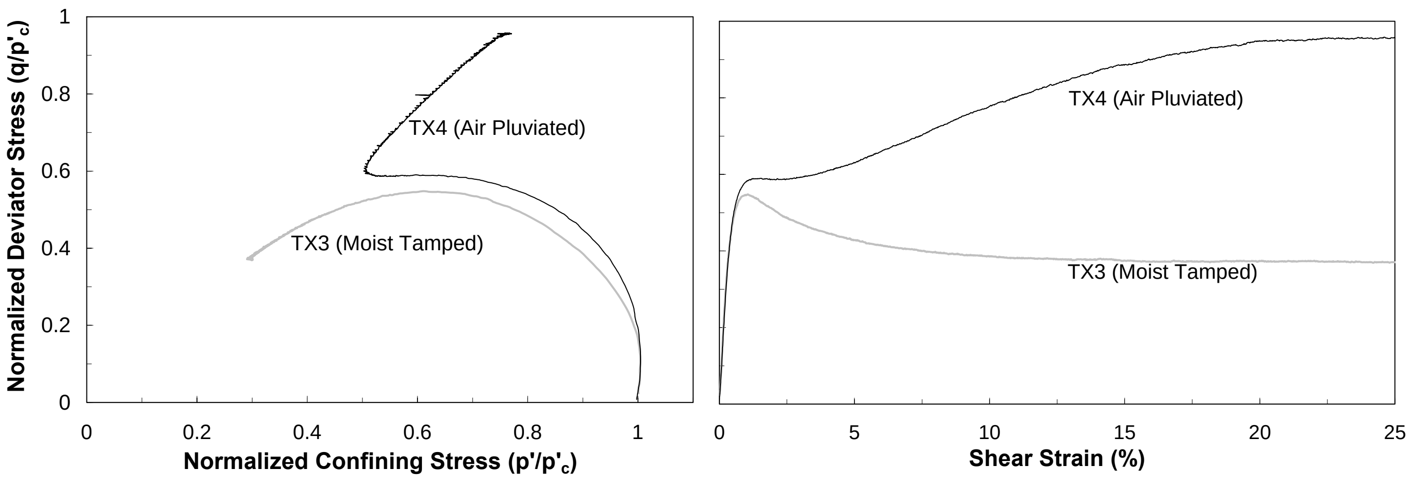

Fig. 3. Normalized stress paths and shear stress – strain plots of triaxial compression tests on specimens of IR sand

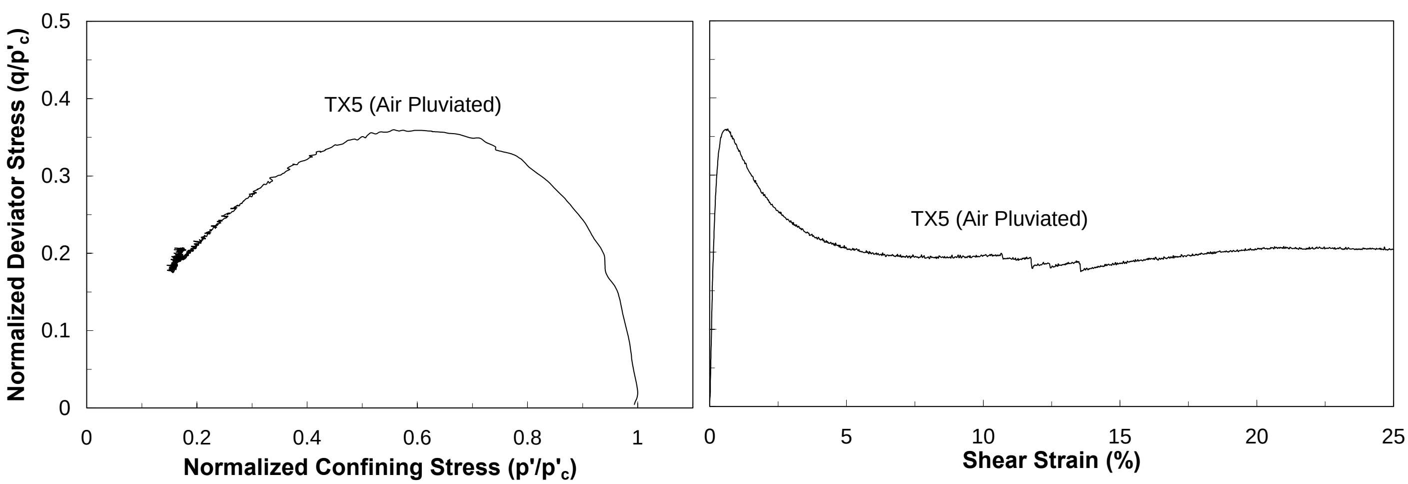

Fig. 4. Normalized stress path and shear stress – strain plots of triaxial compression test on specimen of MR sand

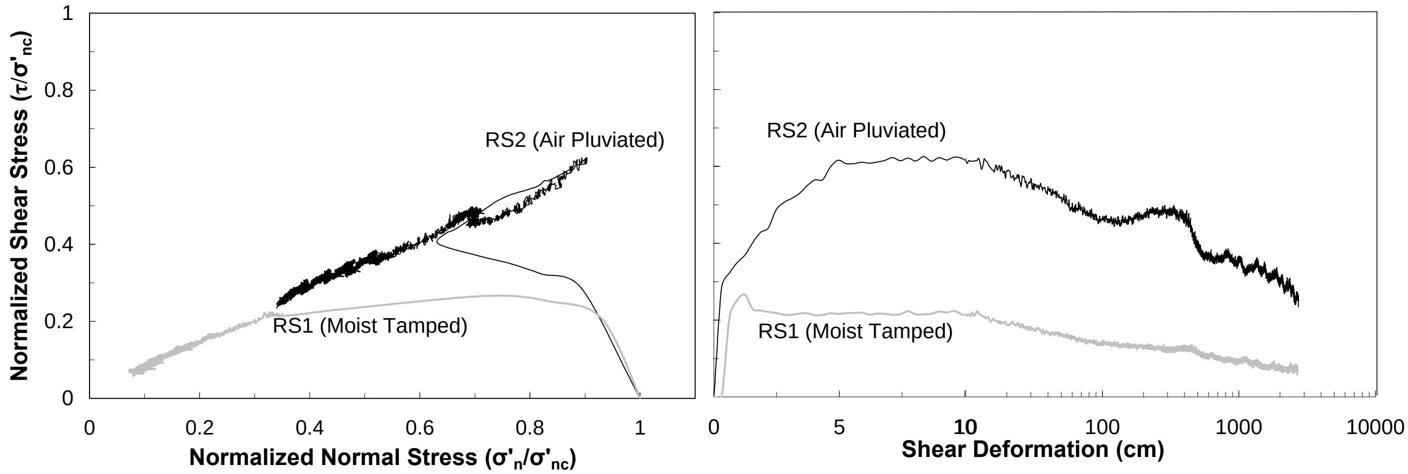

Fig. 5. Normalized stress paths and shear stress – displacement plots of ring shear tests on specimens of CO sand

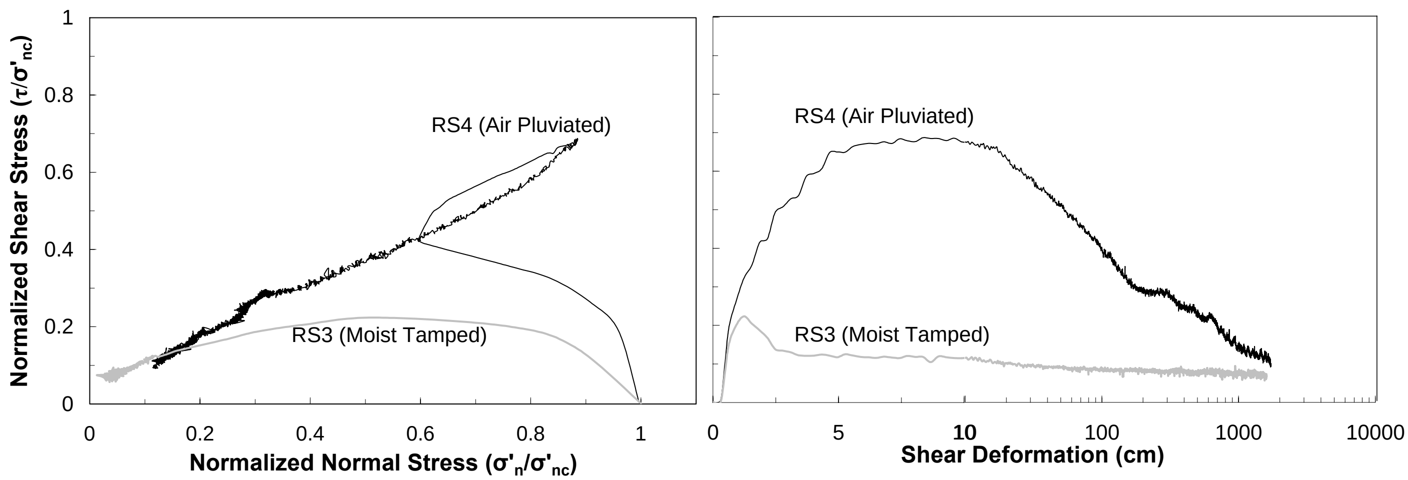

Fig. 6. Normalized stress paths and shear stress – displacement plots of ring shear tests on specimens of IR sand

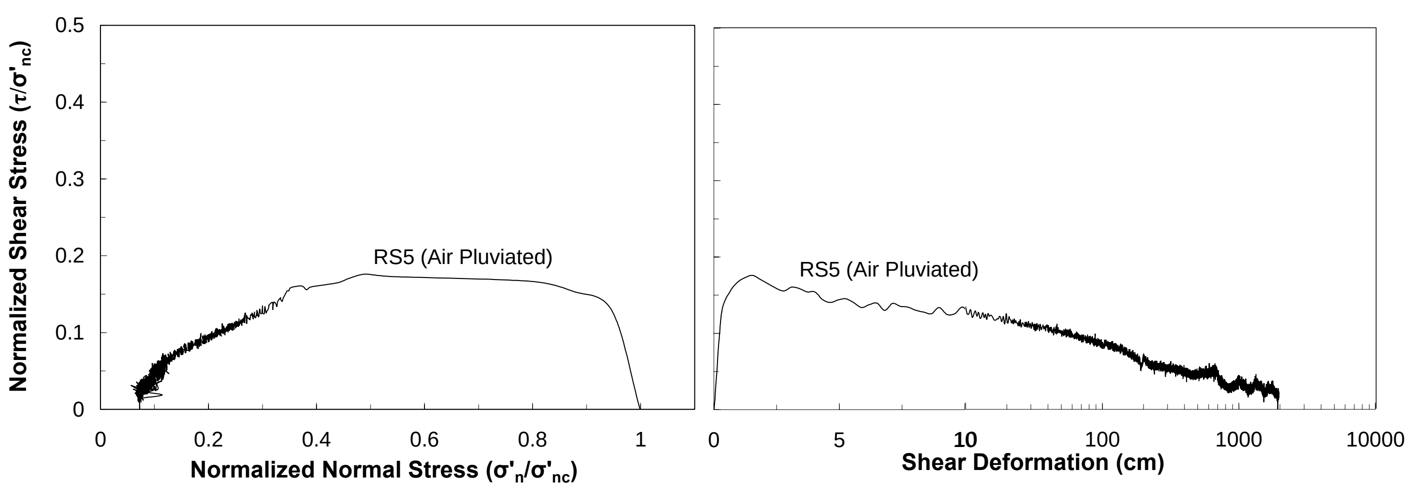

Fig. 7. Normalized stress path and shear stress – displacement plots of ring shear test on specimen of MR sand

TEST RESULTS

Figures 2 through 4 present the stress paths and stress-strain response of the undrained triaxial compression tests performed on moist tamped and air pluviated specimens of CO, IR, and MR sands, respectively. To facilitate comparison, we normalized the shear and confining stresses by the consolidation stress in these plots. As expected, Figs. 2 and 3 illustrate that the loosest possible structure (i.e., void ratio) achieved by moist tamping of CO and IR sands resulted in completely contractive behavior during shear, while the slightly denser samples prepared by air pluviation were only slightly contractive prior to dilating during shear. The air pluviated specimens dilated during shear despite being prepared to the loosest possible void ratio, and exhibiting fairly low relative densities. In contrast, Fig. 4 illustrates that air pluviated MR sand specimen contracted throughout shear, despite having a high initial relative density.

We note that the very low relative densities created with the moist tamping method (particularly for the CO sand) are not likely to develop in natural depositional environments, as discussed by other investigators (e.g., Vaid, 1994; Vaid and Sivathayalan, 2000; Frost and Park, 2003), and that moist tamping is unlikely to reproduce in-situ soil fabrics. Lastly, by comparing grain size distributions measured before and after each triaxial test, we found no evidence of particle crushing of the CO, IR, or MR sands in triaxial compression.

Figures 5 through 7 present the stress path and stress-displacement behavior of moist tamped and air pluviated specimens of CO, IR, and MR sands tested in constant volume ring shear. As with any ring shear test, the shear zone thickness varies during testing making shear strain calculations potentially misleading. As a result, displacement is reported rather than shear strain. Again, we normalized the shear and normal stresses by the consolidation stress to facilitate comparisons.

Again, as expected, Figs. 5 and 6 illustrate that the looser structure (i.e., void ratio) of the CO and IR sands achieved by moist tamping resulted in completely contractive behavior during shear, while the slightly denser samples prepared by air pluviation were only mildly contractive prior to dilating during shear. The air pluviated specimens dilated during shear despite being prepared to the loosest possible void ratio, and having fairly low relative densities. However, at very large shear displacements (which cannot be reached in the triaxial compression tests), both sands contracted. In contrast, Fig. 7 illustrates that air pluviated MR sand specimen contracted throughout shear, despite having a high relative density.

Specimens sheared in the ring shear device formed distinct shear bands, where the shear band thickness ranged from 8 to 10 times the median grain diameter, D_{50} (Sadrekarimi and Olson 2007b). Figures 8 and 9 present grain size distributions of air pluviated IR and CO specimens measured prior to and after shearing (using a portion of the specimen extracted from the shear zone), and clearly show the effect of particle damage and crushing.

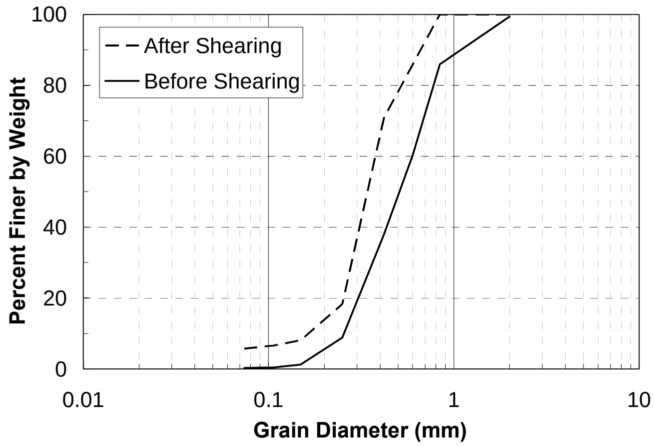

Fig. 8. Grain size distribution of the shear zone before and after the ring shear test on air pluviated IR sand

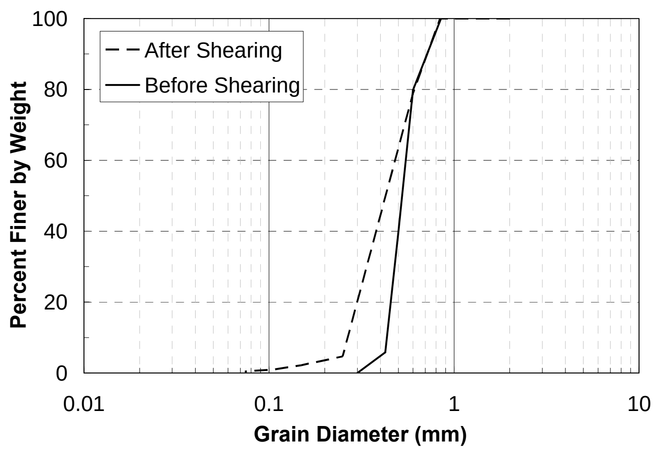

Fig. 9. Grain size distribution of the shear zone before and after the ring shear test on air pluviated CO sand

ROLE OF GRAIN MINERALOGY AND PARTICLE DAMAGE

As discussed earlier, air pluviated MR sand contracted throughout shear, while air pluviated IR and CO sands mildly contracted prior to dilating during shear. This contrasting behavior may, in part, be a result of the different shear moduli of the constituent minerals of each sand. The chief constituent of the MR sand, albite ($NaAlSi_3O_8$), has a shear modulus of 28.6 GPa, while the chief constituent of the IR and CO sands, quartz (SiO_2), has a shear modulus of 44.3 GPa (Ahrens, 1995), more than 50% larger than albite. Therefore, after the initial structure of the soil yields, the albite-rich MR sand should exhibit more compressible behavior than the quartz-rich IR and CO sands, as we observed.

In the case of the MR sand, the total contraction of both particle rearrangement and grain compression is greater than total dilation caused by particle rearrangement, resulting in net contraction. In contrast, the total contraction of the IR and CO sands after initial yield is less than the total dilation, resulting in a net dilative response.

At very large shear deformations achieved in the ring shear experiments, the particles start to become damaged and crush. This particle damage and crushing suppresses the shearing dilation of the IR and CO sands, resulting in a net contractive response and a second phase transformation (from dilative to contractive response) (Sadrekarimi and Olson 2007b).

APPLICATION TO FIELD CASE STUDIES

Most field cases of liquefaction flow failure (e.g. North Dike of Wachusett Dam, USA; Vlietepolder, Netherlands; Helsinki Harbor, Finland; Lake Merced Bank, USA (Olson, 2001); Sheffield Dam, USA (Seed et al., 1969); Hokkaido Dam, Japan (Ishihara et al., 1990); La Palma Dam, Chile (de Alba et al., 1988) have been characterized solely by their gradation, with most the soils that probably liquefied consisting of sandy silts or silty sands. Little or no information is available about their mineralogical composition or any changes in their grain size during shearing.

However, a few flow failure field case histories have some of these details available. For example, the Fraser River Delta slides (McKenna et al., 1992; Chillarige et al., 1997; Christian et al., 1997) and La Marquesa Dam slide (de Alba et al., 1988) involved sandy soils with mixed mineralogies, including non-quartz fines like calcareous and feldspar minerals, and even including clayey fines (Jefferies and Been, 2006). After the failure of the Calaveras Dam, Hazen (1918) reported that the upper parts of the dam that failed consisted primarily of non-siliceous materials cemented with calcium carbonate and calcium sulphate. Similarly, the many flow liquefaction failures that occurred on the banks of the Jamuna Bridge, Bangladesh involved micaeous fine sands with fines content of 2-10% and mica content of 30% (Yoshimine et al., 1999). As another example, the hydraulically-placed Nerlerk berm in Canada experienced numerous flow failures starting during construction in 1983. The soils used for the berm consisted of a silty sand with some clay content. X-ray diffraction showed the Nerlerk berm soil was 84% quartz and 13% feldspar plagioclase (Jefferies and Been, 2006).

Moreover, the Nikawa rapid landslide triggered by the 1995 Hyogoken-Nambu earthquake (which tragically killed 34 people and destroyed 11 houses) involved grain crushing in the shearing zone of the slide (Sassa, 1995). The soil in the shear zone of this landslide consisted of a partially saturated, medium dense to dense coarse-grained limnic or marine clean to silty sand. While the sand (in its initial state) was not readily susceptible to liquefaction, the grains were very prone to crushing (Gerolymos and Gazetas, 2007). Sassa et al. (1996) suggested that grain crushing in the shear zone of this landslide generated large, positive excess pore water pressures, allowing the landslide to travel more than 100m.

Similarly, post-failure field observations confirmed that grain crushing happened in the shear zone of the Hiegaesi Landslide that occurred in southern region of Fukushima Prefecture, Japan. This long run-out landslide was triggered by the heavy rainfalls at the end of August 1998 (Wang et al., 2002). As another example, the Higashi Takezawa and Terano landslides, which were both triggered by the 2004 Mid-Niigata Prefecture earthquake (M6.8), also involved particle crushing. Both of these landslides involved thick Tertiary marine sands overlying stiff siltstone bedrock. In both of these cases, Sassa et al. (2005) speculated that shaking under a large overburden stress facilitated grain crushing and created large, positive excess porewater pressures in the shear zone through the sands.

These cases suggest that the presence of more compressible minerals and grain crushing can sufficiently increase the compressibility of mixed mineral sandy soils to result in net contractive behavior. Therefore, although the initial structure of the soil was medium dense to dense in some of these cases, grain compressibility and crushing generated sufficient contractive behavior and positive porewater pressure generation to cause the failures.

CONCLUSIONS

This study reinforces that the depositional method greatly affects the undrained stress-strain responses of sands at the small to intermediate range of shear strains that can be achieved in the triaxial device. Therefore, it is most appropriate to compare patterns of behavior observed in the triaxial device based upon specimens prepared using the same depositional methods. Similarly, at small to intermediate displacements, the depositional method affects the shear stress-displacement behavior in the ring shear. However, at larger displacements, the original fabric of the soil is erased, and samples prepared using different depositional methods show much smaller differences in shear resistance. The majority of soils that have liquefied historically involve alluvial deposits. Therefore, depositional techniques that simulate these natural depositional processes as close as possible (such as pluviation methods) should be used in laboratory investigations

This study also illustrates the potential effect of grain mineralogy on post-yield stress-strain behavior of coarse-grained soils. Here, two air pluviated sands consisting of chiefly quartz grains exhibit dilative response at intermediate shear strain (and shear displacement) levels. In contrast, an air pluviated silty sand consisting chiefly of more compressible albite grains exhibits contractive behavior at all shear strain (and shear displacement) levels, despite having higher initial relative densities. Similarly, where information is available, it appears unusual that field liquefaction flow failures occur in sands that consist of chiefly quartz minerals. Most of these liquefaction flow failures occur where the sands consist of mixed grain mineralogy, and contain non-quartz fines (e.g., calcareous and feldspar minerals), and even contain some fraction of clayey fines. All of these factors can significantly increase the compressibility of the soil mass.

Lastly, the shearing behavior of a soil is significantly affected by particle damage and crushing at large displacements achieved in the ring shear device. The extent of particle damage and crushing depends, in part, n the shear modulus and strength of the constituting minerals. Grain crushing increases the compressibility of the soil mass, and can lead to net contraction in otherwise dilative soils. Grain crushing may occur in many flow failures and landslides in the field, and has been directly measured in a limited number of landslides.

Thus, a combination of factors, such as grain mineralogy and compressibility, particle damage and crushing, and shear zone formation, may help to explain the often observed discrepancies between laboratory and field behavior during undrained shearing and liquefaction.

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