

STRENGTH AND DEFORMATION CHARACTERISTICS OF RE-DEPOSITED LAHAR FROM MT. PINATUBO

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ABSTRACT

The 1991 eruption of Mt. Pinatubo caused extensive damage, with subsequent lahar flows covering widespread areas. Rehabilitation and reconstruction activities are currently being undertaken, with new buildings and other civil engineering structures being built over lahar-covered areas and roadway/river embankments being constructed using lahar materials. However, there is insufficient knowledge regarding the geotechnical properties of these volcanic deposits, especially their strength and deformation characteristics. To address this issue, an extensive research program was performed in order to investigate the geotechnical characteristics of freshly re-deposited lahar deposits from Mt. Pinatubo and to understand their behavior as materials for geotechnical construction. Freshly re-deposited samples were obtained at two locations: one upstream and another downstream from the volcano. In addition to engineering properties, permeability and compaction characteristics, the strength and deformation behavior in drained condition were investigated using the hollow torsional shear apparatus, with emphasis on the effects of relative density and confining pressure on the shear stress and volumetric change behavior. Moreover, the response of Pinatubo lahar during seismic loading was investigated by determining their cyclic deformation properties and liquefaction resistance. The findings obtained in this study can be used to formulate appropriate design methodology on the use of re-deposited lahar as geotechnical materials.

I. Introduction

The eruption of Mt. Pinatubo in 1991 produced voluminous volcanic products, which have been re-deposited on the slopes of the volcano. These erodible pyroclastic fills were later mobilized by rainfalls in the form of lahar, a rapidly flowing mixture of volcanic ash, debris and water. Subsequent lahar flows have devastated low-lying areas near the volcano, destroying structures along its path, burying widespread areas and back-flooding of debris-choked tributaries. Even now, 11 years after the eruption, these lahar flows still continue to be hazardous for years to come to adjacent communities, many of which were previously populated areas.

Currently, rehabilitation and reconstruction activities are being undertaken in the areas inundated by lahar flows. Buildings and other civil engineering structures are being erected over lahar deposits which cover widespread areas, some by as much as 7m thick. New roads are being built over the devastated areas, while old roads are being elevated to new alignments. Moreover, river embankments, known locally as “mega-dikes”, were constructed with the main purpose of containing the sediments and channel ash flows, thereby protecting the surrounding residential areas and farm lands from the devastation of lahar flow during heavy rainfall. Therefore, due to its abundance and because of economic considerations, efforts are being made to use re-deposited lahar in the affected areas either as construction material for these highway and river embankments or as foundation ground to new buildings. However, there are reports [1] that some portions of the mega-dike system located near the Pasig-Potrero River failed, which was attributed mainly to the lack of knowledge regarding the geotechnical properties of the fill material, which in turn led to insufficient design of such structure.

Although various researches have been conducted to address the problems caused by the eruption of Mt. Pinatubo, most of the initial studies focused mainly on the geological and hydraulic issues [2, 3]. Because of its abundance, investigations have also been conducted on the use of lahar as construction materials in structural engineering, such as aggregates for structural concrete [4] and modular panels for mass housing projects [5] to name a few. However, published researches on the geotechnical properties of these volcanic materials are limited and design methodologies regarding their use as earth embankments are non-existent. For example, the study made by Zarco et al. [6], where they performed both field exploration and laboratory element tests, are among the very few studies made to address the characteristics of lahar from geotechnical point of view.

Therefore, as a supplement to the work done by Zarco et al. [6] and to address the need for a thorough understanding of the geotechnical characteristics of re-deposited volcanic materials, a research program was conducted to determine the engineering properties and strength ~ deformation characteristics of lahar using the hollow torsional shear apparatus. In addition, their dynamic deformation characteristics and liquefaction resistance were investigated. In practical terms, the research also aims to provide engineers with design parameters, guidelines and recommendations on the use of re-deposited lahar from Mt. Pinatubo for geotechnical purposes.

II. The 1991 Pinatubo Eruption

Mt. Pinatubo is a part of a chain of volcanoes which border the western side of Luzon. Located about 100 km northwest of Manila, it is one of the 21 active volcanoes in the Philippines. Mt. Pinatubo (elev. 1745 m) is a predominantly andesitic volcanic cone whose flanks are overlain by massive pyroclastic deposits from a number of eruptive events within the relatively recent geologic past (i.e. the past 5,000 years). At least two eruptive episodes before 1991 have been documented from existing pyroclastic flow deposits surrounding the mountain [7]. The younger of these events is believed to have occurred about 600 years ago, on the basis of carbon dating of plant material contained in the deposits. Each of these two events produced significant volumes of unconsolidated volcanic debris that undoubtedly led to accelerated erosion rates and produced large and frequent lahar flows for several years afterward.

The first visible sign of renewed activity of Mt. Pinatubo were small explosions at its upper northwest slopes on April 1991. This was followed by ash emissions on June 3, and the first eruption occurred on June 9. On June 12, a succession of major explosive eruptions began producing substantial pyroclastic fall and flow deposits on the flanks of the volcano. The most violent explosion occurred in the early afternoon of June 15, ejecting large amount of tephra (volcanic ash and pumice lapilli) and pumiceous pyroclastic flows and forming 20 ~ 40 km high vertical mushroom clouds of ash, steam and other volcanic materials. The passage of a typhoon near the volcano on June 15 aggravated the situation, where the gusty winds generated by the typhoon carried the airborne volcanic materials affecting widespread areas. It also triggered destructive debris flows and also wetted the ash accumulated on roofs which, together with the intense seismic activity accompanying the climactic eruption, resulted in extensive roof collapse, the principal cause of eruption-related deaths. Until September 1991, 218 eruptions were recorded, with the major eruptive episode occurring from June 12-16. Pyroclastic flows and tephra deposits blanketed an area of more than 100 km² around the volcano, greatly altering local drainage patterns and provided large deposits of source materials for subsequent lahar flows. Figure 1 shows the extent of lahar deposition adjacent to the volcano.

Considered as one of the largest volcanic events of this century, the violent eruption of Mount Pinatubo caused rain-induced massive mudflow, sediment deposition and flooding that buried population centers and destroyed roads and bridges, crops, buildings and agricultural lands. An estimated 6 billion cubic meters of pyroclastic material and volcanic ash were deposited over a 4000-square kilometer area including the eight river basins that drain the volcano. The pyroclastic deposits ranged in thickness from a few meters in the valley areas to as deep as 100~200m in river gullies; while the volcanic ash ranged from a few centimeters in areas about 40~50 km from the volcano to about a half-meter near the crater [8]. Although major loss of life from pyroclastic flows was avoided by a successful and timely evacuation of about 60,000 people living within 30 km of

the volcano, up to 800 people were killed and 100,000 became homeless following the eruptions. In addition, millions of tons of sulfur dioxide were discharged into the atmosphere, resulting in a decrease in the temperature worldwide over the next few years.

To incorporate the effect of deposition on the characterization of lahar materials, freshly re-deposited samples were obtained, and the sampling sites are also indicated in Figure 1. Pinatubo-1 sample was collected in the downstream section near the towns of Guagua and Bacolor. Pinatubo-2 sample, on the other hand, was collected in the upstream section near the location of the former U.S. Clark Air Base. The materials collected were disturbed samples which were later reconstituted in the laboratory for testing.

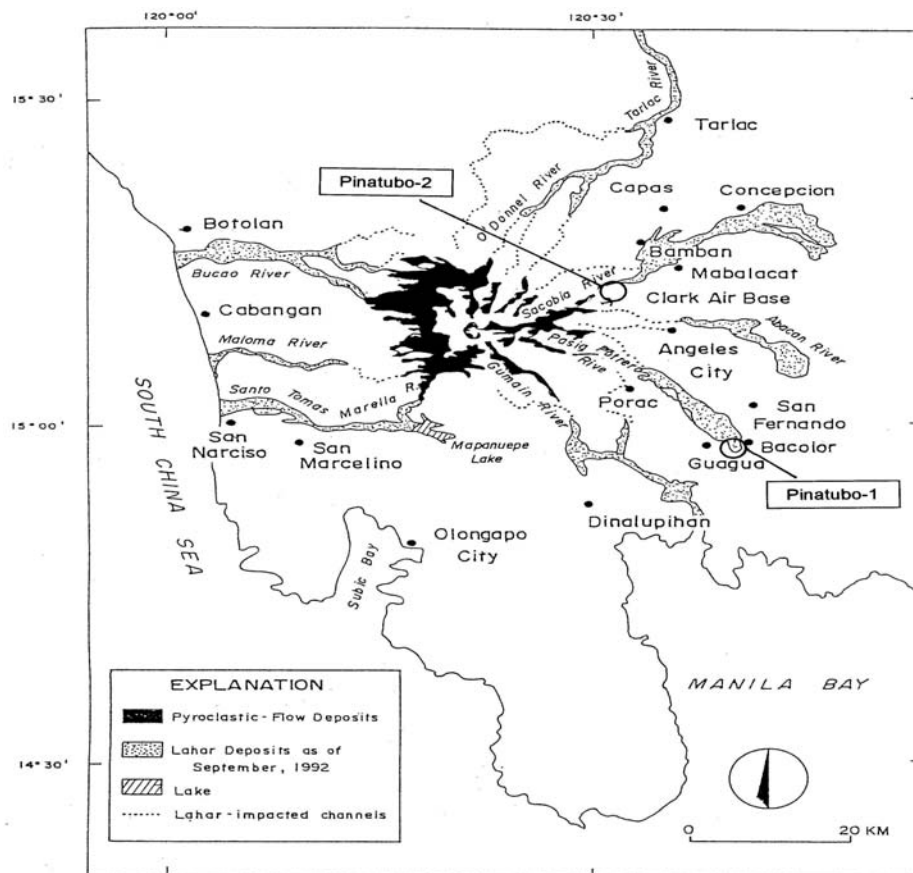


Figure 1. Map showing the lahar coverage near Mt.Pinatubo [2] and the sampling sites.

III. Grain Size Distribution and Other Index Properties

In the torsional shear tests performed in this study, a hollow torsional shear apparatus was employed. In these tests, only the particles passing through the 2 mm sieve were used to prepare the specimens for testing due to the limitation of the equipment. It is worth mentioning that all the tests were performed according to the JGS (Japanese Geotechnical Society) standards.

The results of the grain size analysis on the sieved materials (after particles coarser than 2 mm were removed) for the two samples are shown in Figure 2. It can be seen that both samples are poorly graded with little fines ($F_c = 1.43\%$ for Pinatubo-1 and $F_c = 4.68\%$ for Pinatubo-2).

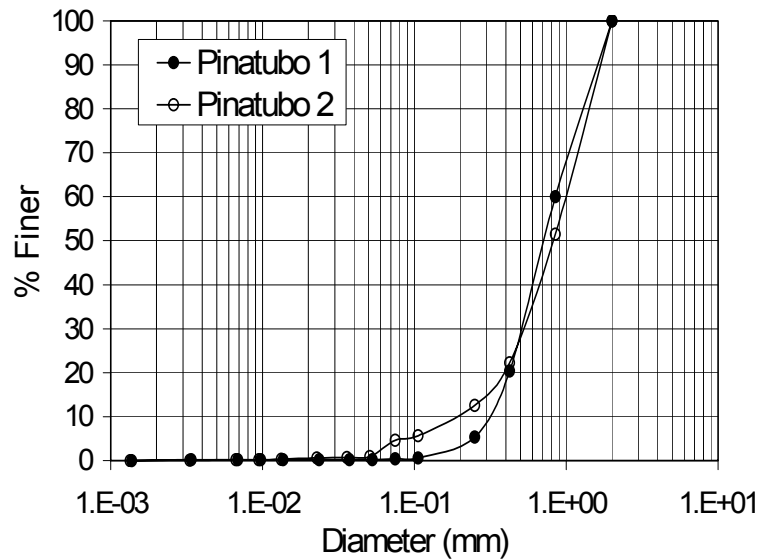


Figure 2. Grain size distribution curves after particles coarser than 2 mm were sieved out.

The index properties of these materials are shown in Table 1. For each sample, specific gravity tests were performed six times and the average was taken to represent the specific gravity G_s of the said sample. Moreover, the tests for obtaining the minimum and maximum void ratios (e_{min} and e_{max}) were repeated at least fifteen times to ascertain the consistency of the results. It is noted that these index properties are consistent with the typical values for granular materials. Moreover, microscopic images of Pinatubo-1 and Pinatubo-2 samples are shown in Figure 3 where it can be seen that the particle shapes are generally sub-rounded.

Table 1.
Index properties of re-deposited lahar

	Pinatubo-1	Pinatubo-2
Specific Gravity, G_s	2.67	2.74
Max. Void Ratio, e_{max}	0.918	0.929
Min. Void Ratio, e_{min}	0.699	0.629

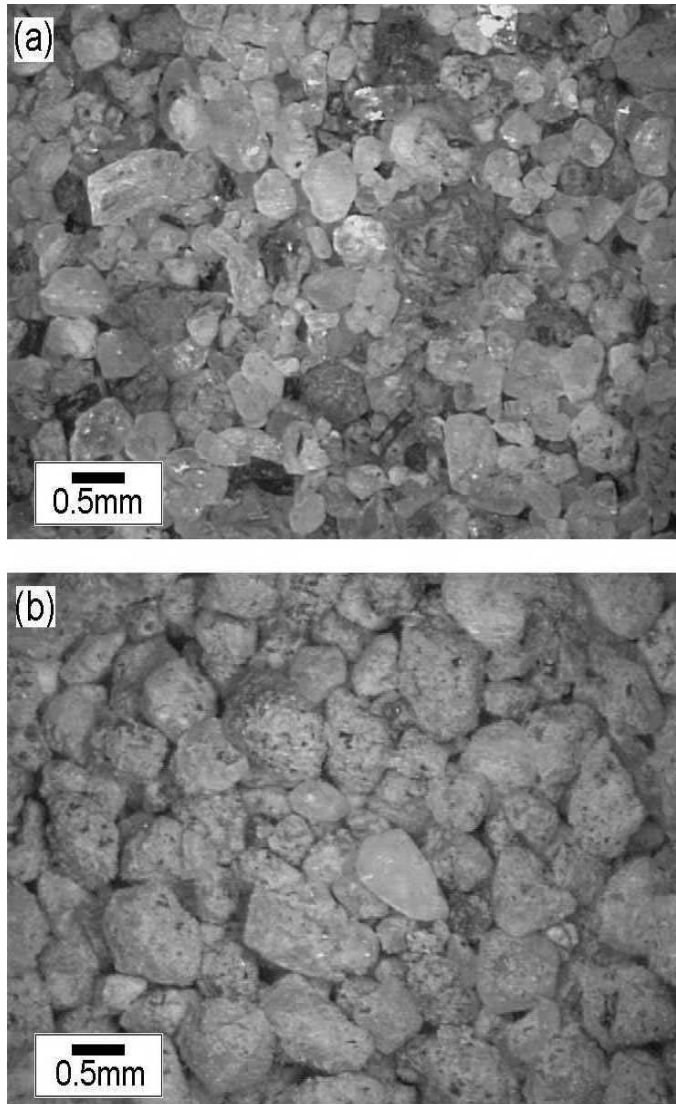


Figure 3. Microscopic image of (a) Pinatubo-1; and (b) Pinatubo-2 samples.

IV. Compaction and Permeability Characteristics

Figure 4 shows the results of the compaction tests conducted on both samples. Note that although the index properties and grain size characteristics of these two materials are almost the same, their compaction characteristics are very different. This may be due to the fact that the grains of Pinatubo-2 sample are crushable, as proven in subsequent investigations, and this makes it more compactible than Pinatubo-1 sample. Similar observation was made by Morales [9] on lahar samples taken near Mt. Pinatubo where he noted that the cemented pozzolanic materials were dis-aggregated with mechanical pounding which thereby resulted in a higher density.

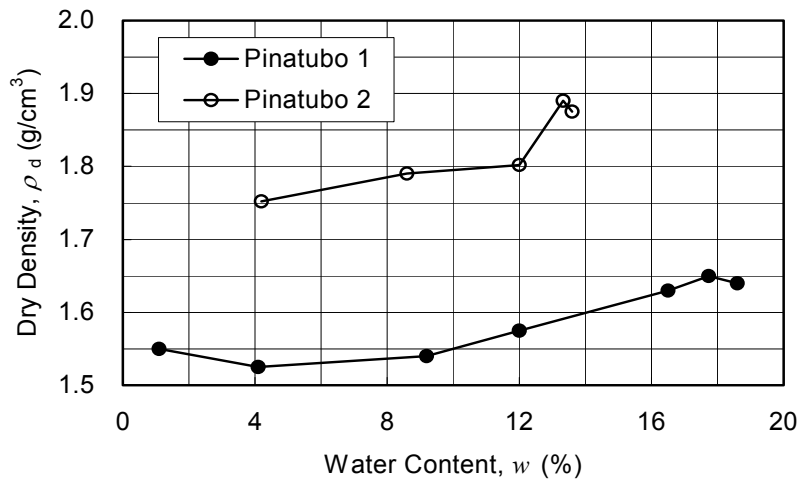


Figure 4. Compaction curves.

Next, the permeability characteristics of the lahar samples were investigated. The permeability of soil is a measure of its resistance to water flowing through it and is quantified by the coefficient of permeability k . Constant head permeability tests were performed to determine the coefficients of permeability at different relative densities. The results are shown in Figure 5. Note that the permeability of granular soil depends mainly on the cross-sectional areas of the pore channels, which can be expressed in terms of effective grain size (usually, the mean diameter corresponding to 10% passing, i.e. D_{10} , is used). Pinatubo-1 sample is more permeable because it has larger D_{10} ($= 0.30$ mm) than Pinatubo-2 ($D_{10} = 0.19$ mm) sample. Nonetheless, the permeability characteristics of these volcanic soils are consistent with those observed for typical granular soils [10].

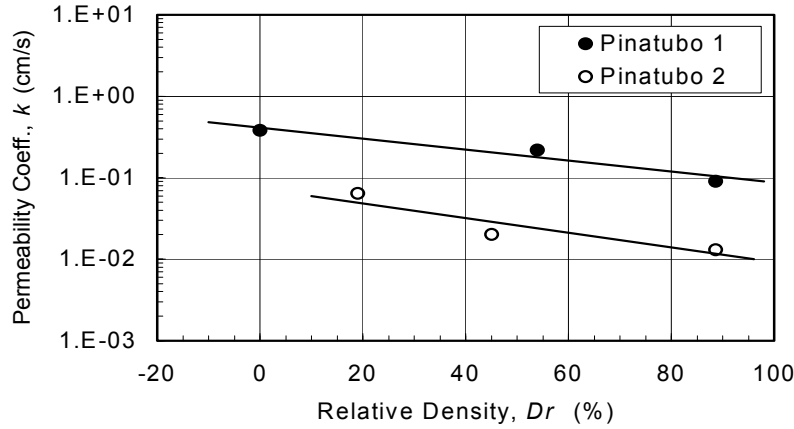


Figure 5. Permeability characteristics

V. Strength and Deformation Characteristics

5.1 Apparatus

To investigate the strength and deformation properties of re-deposited lahar, several series of monotonic drained shear tests were performed. For this purpose, the hollow torsional shear apparatus was employed. A cross-section of the apparatus is shown schematically in Figure 6(a). As mentioned earlier, only the particles passing through the 2 mm sieve were considered in the tests due to the limitation of the apparatus. The hollow cylindrical specimen, with an inner radius $r_i = 30$ mm, outer radius $r_o = 50$ mm and height $h = 195$ mm, was enclosed laterally by two flexible membranes and vertically by rigid top and bottom caps. The loading system enabled the independent control of the torque, the axial load, the inner cell pressure and the outer cell pressure. The torsional and axial loads were applied using Bellofram cylinders and were measured by a load-torque cell located inside the cell chamber. The outer and inner cell pressures were controlled independently and were both measured accurately by two high-capacity differential pressure transducers. Similarly, the volume changes of the sample and inner cell were measured by two low-capacity differential pressure transducers that convert the height of water in the burette to water pressure. Typical induced stresses in the specimen are depicted in Figure 6(b). A more detailed discussion of the shear apparatus is given elsewhere [11].

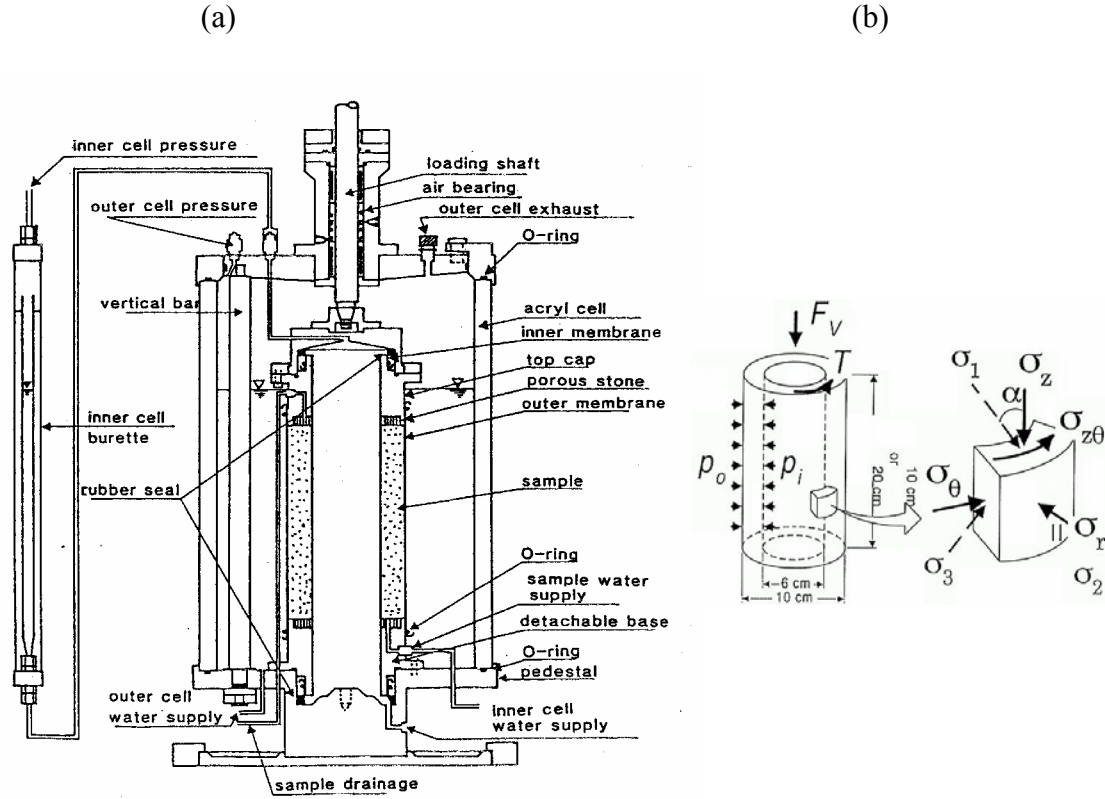


Figure 6. Hollow cylindrical triaxial apparatus: (a) schematic diagram; and (b) induced stresses.

5.2 Test Procedure

To ensure uniform samples throughout the whole experimental program, the samples were prepared by air pluviation method, which is ideal for sandy materials with little or no fines. In this method, air-dried sand was continuously pluviated in the mold from a plastic container with a tube attached at its bottom part. The pluviation was carefully done while maintaining the height of the pluviation (distance between the bottom end of the tube and the sand surface) constant until the mold was filled with sand. Then, the excess material was carefully removed and the upper surface of the specimen was smoothed and formed at the prescribed height of $h = 195$ mm. The density of the sample was controlled by adjusting the height of the pluviation. A larger drop height results in a denser sample.

In order to ensure a high degree of saturation of the samples, carbon dioxide (CO₂) was circulated through the sample for approximately 1.5hr. Then, de-aired water was slowly introduced into the sample through the bottom cap line under a very low static force (4 kPa). Saturation was achieved when a sufficient amount of water had percolated through the specimen. Back pressure was applied to the sample and it was allowed to stand overnight. Following this procedure, a B-value of at least 0.95 was achieved. Note that this technique resulted in satisfactory B-values compared to those obtained by Zarco et al. [6] in their triaxial tests where the B-values of the test specimens never exceeded 0.85. After full saturation, the sample was then isotropically consolidated to final mean effective stress of $p' = 29.4, 98, 196$ or 294 kPa. In all the tests, the sample was allowed to consolidate for one hour.

In the monotonic drained tests, the sample was sheared under drained conditions by applying torque at a rate of approximately 18%/min. All the normal stress components (see Figure 6(b)) remained constant during shearing, i.e. $\sigma'_z = \sigma'_r = \sigma'_\theta = \text{constant}$, and consequently, the mean effective stress was constant. The samples were sheared until a shear strain of about $\gamma_{at} = 20\%$ was attained. Stress changes were not applied until the sample had ceased straining from the preceding stress increment.

5.3 Influence of Initial Relative Density and Mean Effective Confining Pressure

In traditional soil mechanics, the characteristics of soil behavior have always been associated with the initial densities as well as the confining pressure of the soil. To characterize the strength and deformation behavior of each lahar sample, a series of monotonic drained torsional shear tests was conducted on samples with varying relative densities under constant mean effective confining pressure, or vice versa. The results of tests for each sample are discussed below.

Pinatubo-1 Sample

The influence of relative density on the drained torsional shear behavior of Pinatubo-1 sample is illustrated by the results of tests conducted on samples with relative density $Dr = 24, 53$ and 80% at mean effective confining pressure $p' = 98$ kPa. All the relative densities mentioned herein refer to those after consolidation but before shearing. The normalized stress-strain and volumetric change behavior for this series of tests are shown in Figure 7. In the figure, the stress τ is normalized by the mean effective confining pressure p' and given by the stress ratio τ/p' , while the volumetric change is expressed in terms of the volumetric strain, ε_v . Note that $+\varepsilon_v$ denotes compression while $-\varepsilon_v$ refers to dilation. It can be seen that as Dr increases, the stiffness of the normalized stress-strain curve and the peak stress ratio increase. Consequently, the stiffest and the strongest behavior is exhibited by the sample prepared at $Dr = 80\%$ while the softest and weakest behavior is shown by the sample prepared at $Dr = 24\%$. From

the volumetric change behavior shown in Figure 7(b), denser sample exhibits weak compression followed by strong dilation while loose sample shows the opposite. The behavior is predominantly dilative and a large difference in dilatancy can be observed for different densities.

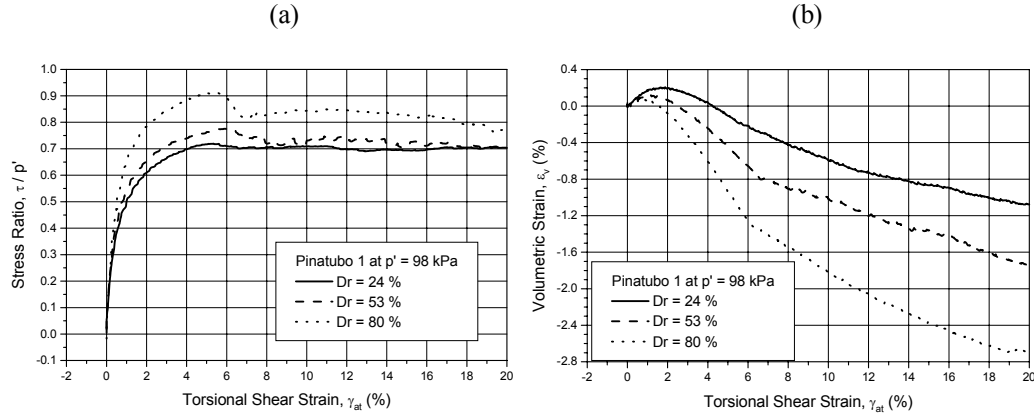


Figure 7. Influence of relative density on the (a) stress-strain and (b) volume change behavior of Pinatubo-1 sample at $p' = 98$ kPa.

In a similar manner, the effects of confining pressure are illustrated through a comparison of the results obtained in the tests at different mean effective confining pressures p' , on samples with almost the same density prior to shearing. The normalized stress-strain curves and volumetric strain-shear strain relationship for Pinatubo-1 samples with a relative density of $Dr = 50\sim 53\%$ obtained in the tests with $p' = 29.4, 98$, and 196 kPa are shown in Figure 8. The stiffness of the response decreases with increasing mean effective confining pressure, causing a downward shift of the stress-strain curve associated with a reduction in the maximum stress ratio. Moreover, the volumetric strain-shear strain relations are predominantly dilative and increasing the confining pressure has the effect of increasing the compression and weakening the dilation.

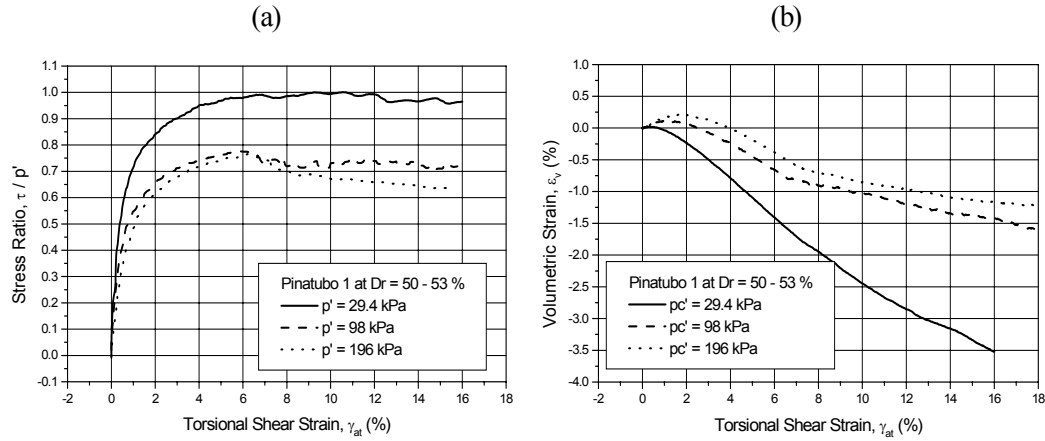


Figure 8. Influence of mean effective confining pressure on the (a) stress-strain and (b) volume change behavior of Pinatubo-1 sample with $Dr = 50 \sim 53\%$.

Note that these patterns of response with respect to changes in relative density and mean confining pressure are consistent with the observations made by various researchers (e.g., [12]~[14]) on the strength and deformation characteristics of granular materials.

Pinatubo-2 Sample

A series of tests was conducted on Pinatubo-2 samples prepared at a fixed relative density $Dr = 59 \sim 70\%$ and subjected to various levels of confining pressure $p' = 29.4, 98$ and 196 kPa. The normalized stress-strain and volumetric strain-shear strain relations are shown in Figure 9. Similar to the observations made on Pinatubo-1 samples, the normalized stress-strain curve for Pinatubo-2 sample becomes stiffer and the peak stress ratio becomes higher when the confining pressure is reduced. However, unlike Pinatubo-1 sample, the volumetric strain-shear strain relationship is predominantly compressive and the amount of compression increases as the confining pressure increases.

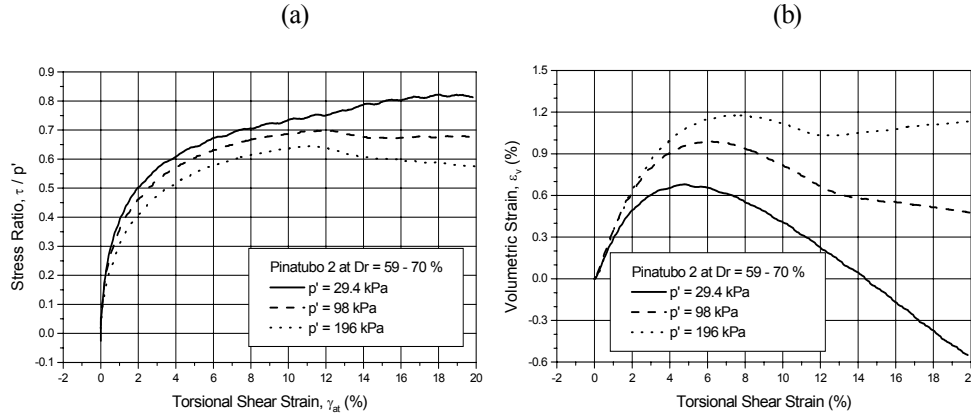


Figure 9. Influence of mean effective stress on the stress-strain and volume change behavior of Pinatubo-2 sample with $Dr=59\sim70\%$.

Comparing the normalized stress-strain and volumetric change behavior of the two materials at a specific condition (e.g., $p' = 98$ kPa and $Dr = 55\%$), it can be observed that Pinatubo-1 sample exhibit stiffer stress-strain response, higher peak stress ratio and more dilative volumetric behavior than Pinatubo-2 sample. It also shows higher residual shear stress ratio, although the difference is less obvious. In addition, Pinatubo-1 sample shows a more obvious strain softening response after the peak shear stress is achieved. Pinatubo-2 sample, on the other hand, exhibits weaker behavior and contractive volumetric strain response.

For each sample, the strength parameters based on the Mohr-Coloumb failure line were derived by constructing the Mohr's circle of stresses corresponding to the peak and the residual states. The derived strength parameters for medium dense samples ($Dr = 50\sim70\%$) are summarized and tabulated in Table 2. From the table, it can be clearly seen that Pinatubo-1 sample is the stronger material. Note that both materials seem to take on quite large values of apparent cohesion, c , and the values are much higher in the residual state compared to the peak state.

Table 2.
Strength Parameters.

Sample Name	State	Friction Angle, ϕ (deg)	App. Cohesion, c (kPa)
Pinatubo-1 ($Dr = 50\sim53\%$)	peak	46.0	8.2
	residual	36.0	13.2
Pinatubo-2 ($Dr = 59\sim70\%$)	peak	38.3	6.3
	residual	32.9	11.0

Effects of Particle Crushing

When sands are subjected to high confining pressure, particles are sometimes crushed. The crushing, in turn, generally affects the stress-strain and volumetric strain behavior of sand. Therefore, it is necessary to confirm whether crushing of particles occurred in the present series of tests. For this purpose, a fresh sample was sheared under monotonically increasing load and the grain size distributions before and after the test were obtained to determine the change in the grain size characteristics. Note that crushing can be detected by looking into the reduction of the grain sizes after shearing. To determine the effect of crushing on the stress-strain behavior, the “used sample” was prepared in the same condition as before and sheared again in the same way as the “fresh” sample and the resulting stress-strain curves were compared.

A couple of tests were performed on each specimen made up of Pinatubo-1 and Pinatubo-2 samples to determine whether they were crushable or not. Each set of tests was performed at similar condition. Although not shown here due to space limitations, the test results indicated that Pinatubo-1 was not crushable since there were no significant changes in the grain size distributions before and after the test. Moreover, there was no meaningful variation in the normalized stress-strain and volumetric change behavior for the “fresh” and “used samples”. On the other hand, tests on Pinatubo-2 samples showed otherwise. A closer inspection of the grain size distribution curves, illustrated in Figure 10(a), reveals that there is a decrease in the weight of large-diameter particles and a subsequent increase in the weight of small-sized particles. This phenomenon is indicative of particle crushing. Also, the stiffness of the stress-strain curve of the “used” sample, illustrated in Figure 10(b), is appreciably less than that of the “fresh” sample. Again, this degradation of the stiffness indicates that the particles have crushed. The evidence of crushing is also reflected significantly in the volumetric change behavior, as depicted in Figure 10(c). The “used” sample is twice more compressive than that of the “fresh” one. Based on the foregoing observations, it was concluded that Pinatubo-2 sample is crushable. This observation is consistent with the findings of other researchers (e.g., [9]) that this material is easily degraded or reduced in size during compaction.

Note that some useful information regarding the performance of crushable volcanic soils as embankment materials can be obtained from the foregoing discussions. The change in the grain size distribution of Pinatubo-2 sample due to crushing is very small, indicating that very few percentage of materials have crushed. However, the degradation of the stiffness of the stress-strain curve and the increase in the compressibility of the soil are quite appreciable. Hence, during embankment construction, the grains of the volcanic soils may sometimes be crushed due to compaction and this could detrimentally

affect its strength. Thus, certain allowance should be made on the strength of crushable volcanic soil when designing embankments and other similar structures made of such materials.

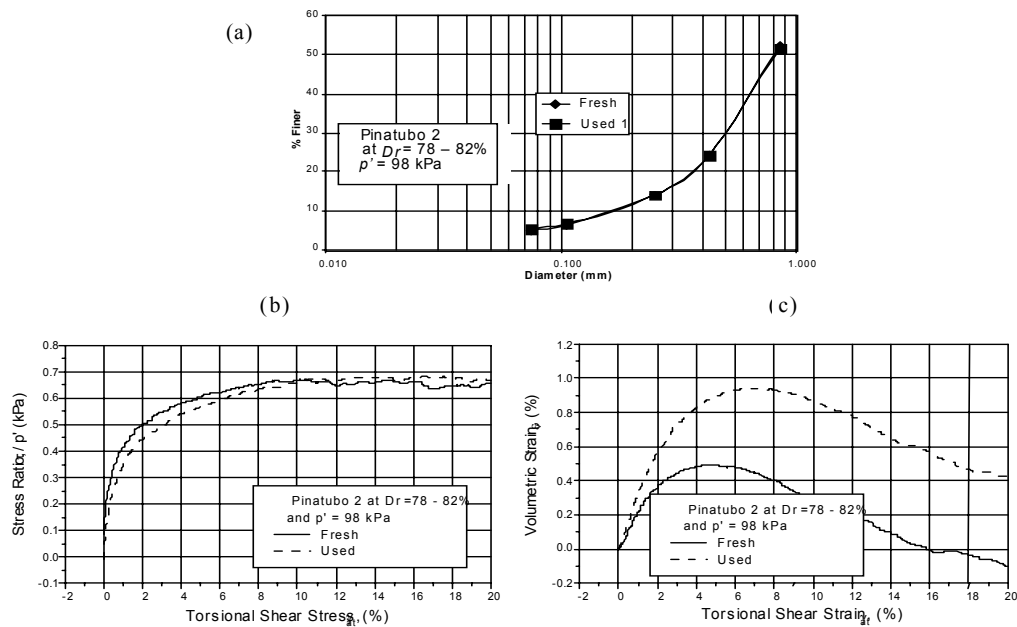


Figure 10. Comparison between the (a) grain-size distribution curves; (b) normalized stress-strain and (c) volumetric change behaviors of the fresh and used Pinatubo-2 samples at $D_r = 78 \sim 82\%$ and $p' = 98$ kPa.

VI. Cyclic Properties of Pinatubo Deposits

A very important consideration in the design and analysis of embankments and foundations is the stability of the structure during an earthquake shaking, including liquefaction. In investigating the behavior of the ground or soil structures during earthquakes, seismic response analysis based on the theory of wave propagation or on finite element principle is usually performed. For this purpose, the cyclic deformation properties and liquefaction resistance of the soil obtained from undrained cyclic torsional tests are very useful.

To investigate the dynamic deformation properties and liquefaction characteristics of re-deposited lahar materials, several series of undrained cyclic torsional tests were performed. The samples were prepared at relative densities $D_r = 24 \sim 27\%$ (liquefaction tests) and $D_r = 53\%$ (dynamic tests) for Pinatubo-1 sample and $D_r = 77 \sim 79\%$ for Pinatubo-2 sample. These relative densities were based on the resulting bulk after the samples were pluviated at zero height. The

big disparity on the relative densities between the two materials may be attributed largely to the pluviation characteristics of the materials and the decrease in void ratio after consolidation. As mentioned earlier, Pinatubo-2 sample is very compressible; thus, a relatively large quantity of water drained out during the consolidation phase resulting in a dense sample.

6.1 Dynamic Deformation Characteristics

A series of undrained cyclic torsional shear tests was conducted to investigate the dynamic properties of Pinatubo samples. Unfortunately, due to the limitation of the hollow torsional shear apparatus, $\gamma_a = 0.01\%$ was the smallest shear strain achievable in all the tests conducted. Figure 11 shows the plots of the normalized secant modulus G/G_0 and damping ratio, D , versus the amplitude of shear strain γ_a for Pinatubo-1 sample, while Figure 12 illustrates similar plots for Pinatubo-2 samples. In the plots, the shear modulus is normalized by the initial shear modulus, G_0 , defined as the shear modulus at $\gamma_a = 10^{-6}$. Since the testing apparatus was incapable of imposing cyclic loading at this level of strain, G_0 was estimated from the drained monotonic test data presented earlier. The first two data points in the stress-strain curve having strains ranging from $\gamma_a = 1 \times 10^{-5} \sim 4 \times 10^{-5}$ were considered in the calculation. Accordingly, the shear moduli for Pinatubo-1 and Pinatubo-2 samples were approximated to be $G_0 = 36.2$ MPa ($Dr = 53.2\%$ and $p' = 98$ kPa) and 22.4 MPa ($Dr = 79.4\%$ and $p' = 98$ kPa), respectively.

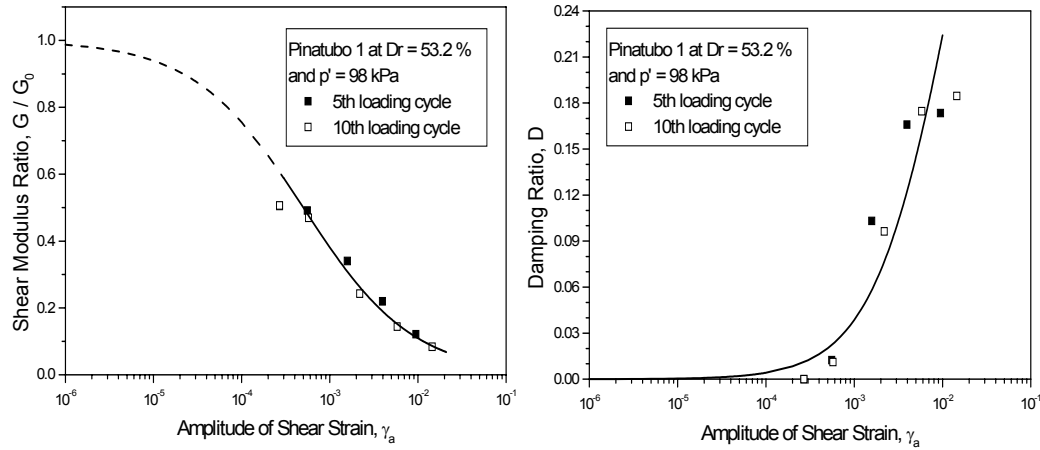


Figure 11. Normalized shear modulus and damping ratio versus shear strain for Pinatubo-1 sample at $Dr = 53.2\%$ and $p' = 98$ kPa.

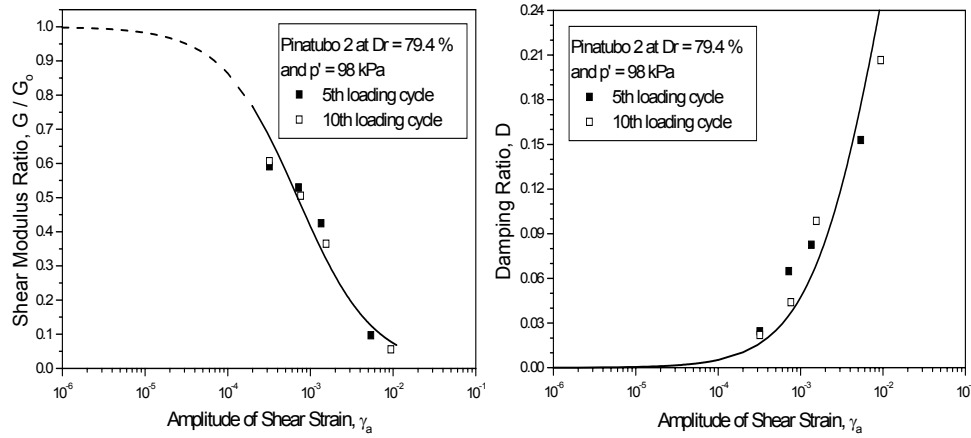


Figure 12. Normalized shear modulus and damping ratio versus shear Strain for Pinatubo-2 sample at $Dr=79.4\%$ and $p'=98\text{kPa}$.

Comparing these two figures, it can be observed that there is quite a small difference between the rate of reduction in shear modulus with respect to shear strain for Pinatubo-1 and Pinatubo-2 samples. For smaller strains ($\gamma_a < 2 \times 10^{-3}$), the rate of reduction of Pinatubo-1 sample is a little smaller than that of Pinatubo-2 sample while the opposite is observed for larger strains ($\gamma_a > 2 \times 10^{-3}$). Generally, Pinatubo-2 sample has higher damping ratio D than Pinatubo-1 sample. Nevertheless, it can be surmised that the dynamic deformation properties of Pinatubo deposits are similar to those typically observed on granular materials (e.g., [15]~[16]).

6.2 Liquefaction Characteristics

To investigate their liquefaction resistance, undrained cyclic torsional shear tests were conducted on reconstituted samples of Pinatubo-1 and Pinatubo-2 samples. In these tests, the samples were sheared cyclically at various levels of stress ratios. In spite of the differences in relative densities, a number of interesting observations can be made from the results of the tests.

Figure 13 shows the cyclic undrained stress-strain behavior of Pinatubo-1 sample at $Dr = 26.8\%$ and Pinatubo-2 sample at $Dr = 79.4\%$. The two samples were cyclically sheared under similar effective confining pressure $\sigma'_c = 98 \text{ kPa}$ and torsional shear amplitude $\tau_{max} = 25 \text{ kPa}$. Figure 14, on the other hand, shows the accompanying stress paths corresponding to the above-mentioned tests. The figures show that in spite of the fact that Pinatubo-1 sample was prepared at a much lower relative density, it exhibited stronger cyclic resistance than Pinatubo-2 sample. Moreover, the denser Pinatubo-2 sample was observed to be the weaker material. Despite its higher relative density, it needed lesser number of cycles to reach 5% double amplitude of shear strain compared to Pinatubo-1 sample. This behavior may be attributed largely to the compressive nature of

Pinatubo-2 sample. From the results presented earlier, during the state when Pinatubo-1 sample was dilating quite enormously, Pinatubo-2 sample was still undergoing compression. Due to the compressive nature of Pinatubo-2 sample, a rapid build up of pore water pressure was observed, as shown in Figure 14. Similar tests on Pinatubo-1 and Pinatubo-2 samples (i.e. same relative density and confining pressure with the earlier tests) but with various maximum amplitude of shear stress, τ_{max} , showed practically similar trends.

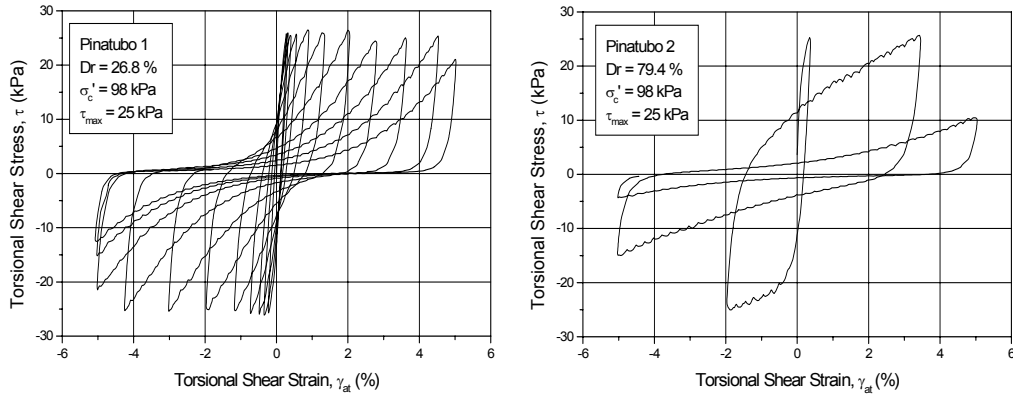


Figure 13. Undrained stress-strain behavior of Pinatubo samples at $\tau_{max} = 25$ kPa.

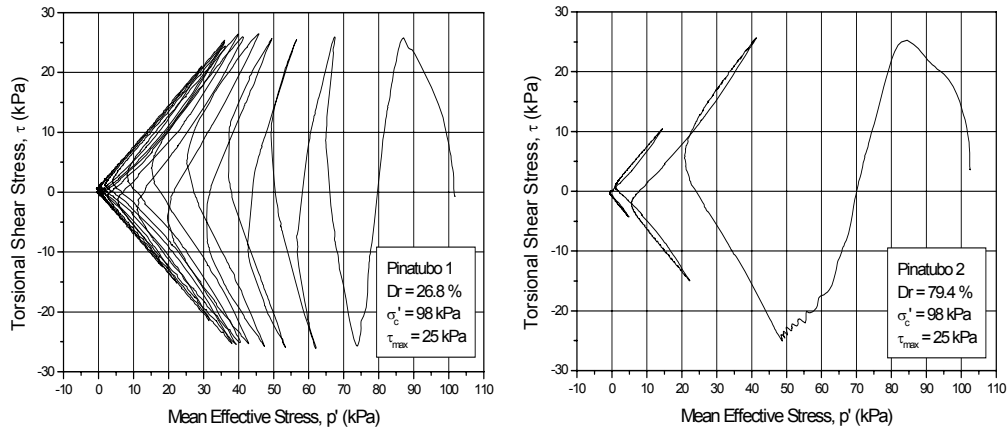


Figure 14. Stress paths of Pinatubo samples at $\tau_{max} = 25$ kPa.

The liquefaction curves for Pinatubo-1 and Pinatubo-2 samples, defined for both the development of 5% double amplitude shear strain and 95% pore water pressure development, are given in Figure 15. It can be clearly seen from the figure that Pinatubo-2 sample has considerably weak resistance against liquefaction despite the fact that it was prepared at relatively denser condition ($Dr = 77\sim 80\%$). Pinatubo-1 sample, on the other hand, was prepared at a very loose condition ($Dr = 23\sim 27\%$) but it exhibited stronger resistance. The cyclic stress ratio (τ/σ'_c) at 20 cycles for Pinatubo-1 sample ($Dr = 23\sim 27\%$) is read off from Figure 15 as being equal to 0.205 while that of Pinatubo-2 sample ($Dr = 77\sim 80\%$) is 0.125. Note that this is true whether liquefaction is defined in terms of 5% double amplitude shear strain or the generation of 95% pore water pressure.

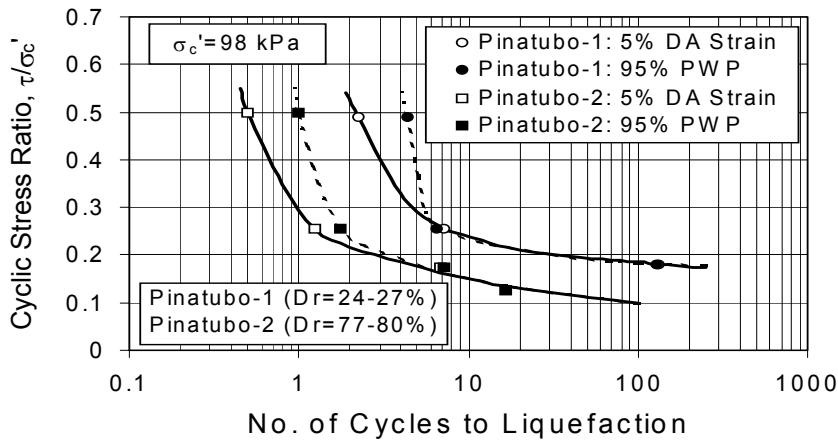


Figure 15. Liquefaction curves of Pinatubo-1 and Pinatubo-2 samples.

This seemingly “strange” observation when viewed from the fact that denser materials have generally higher liquefaction resistance [17] can be explained by looking into the volumetric change behavior of the two materials during the drained monotonic tests presented earlier. In medium dense condition ($Dr = 50\sim 70\%$) and mean effective confining pressure $p' = 98$ kPa, Pinatubo-1 sample exhibited a very dilative behavior (see Figure 8) while Pinatubo-2 sample was very compressive (see Figure 9). During the cyclic shear, Pinatubo-2 sample, being a compressible material, experienced very large negative dilatancy which hastened the occurrence of liquefaction. Dilative materials such as Pinatubo-1 sample, on the other hand, demonstrated quite a good resistance against liquefaction since its tendency to dilate minimized the build up of pore water pressure.

VII. Summary of Major Findings and their Applications

The foregoing discussions on the geotechnical properties of re-deposited lahar taken from the 1991 Mt. Pinatubo eruption show interesting results. Differences in the compaction, permeability, strength and deformation characteristics as well as liquefaction resistance can be observed for samples coming from the same volcanic source but procured from different sampling sites. The same observations were made by Zarco et al. [6] in their investigation of re-deposited lahars from 18 test pits scattered all over the affected area. Such differences may be attributed to the mechanism of deposition (debris flow, lahar flow or pyroclastic flow) which rendered the gradation and characteristics of the upstream different from that of downstream. Samples taken downstream showed very good strength, dilative tendency and fairly adequate liquefaction resistance; on the other hand, samples procured upstream showed lower strength, compressive behavior and high susceptibility to liquefaction, even in dense condition. Also upstream samples showed very crushable behavior, which may be advantageous during compaction operation. Therefore, care must be taken in the selection of re-deposited lahar which will be used as construction materials.

The laboratory tests performed to determine the index properties of lahar reveal results consistent with those observed for typical granular materials. The specific gravity, G_s , vary between 2.67~2.74 while the void ratio ranges from 0.629~0.929. From these values, in-situ unit weights of lahar can be approximated to be similar to ordinary sand.

As in other granular soils, the coefficient of permeability of lahars decreases with increase in relative density. However, even in a compacted state ($Dr = 90\%$), the coefficient of permeability is still relatively high. Piping and erosion may therefore occur when these materials are used as earth embankments. When used in the construction of earth dikes, impervious core, cut-off trenches, interior drainage system, etc. should be provided to minimize seepage. Moreover, slope protection against erosion, such as riprap, soil cement, concrete facing, vegetation, etc., should be employed.

The angles of internal friction ϕ determined from monotonic drained tests are also consistent with those observed for typical medium dense granular materials. These range from 38~46 degrees at the peak state and 33~36 degrees at residual state. Although the tested materials had very little fines content, they tended to exhibit quite large values of apparent cohesion, c . Moreover, the values are much higher in the residual state (11~13 kPa) compared to the peak state (6~8 kPa).

From the results of monotonic drained shear tests, it is observed that lahar deposits show generally similar pattern as those observed in typical granular materials. The stiffness of the stress-strain response and the peak stress ratio decrease with increasing mean effective confining pressure causing a downward shift of the curve. The volumetric change relationship is characterized by a gradual change in the behavior, from being compressive with weak dilation for high confining pressure and being expansive with strong dilation for low confining pressure. On the other hand, the effects of decreasing the relative density are characterized by a decrease in the stiffness of the stress-strain response and in the peak stress ratio while the volumetric change behavior becomes more compressible and weaker in dilation.

Materials represented by the Pinatubo-1 sample exhibited higher peak stress ratios than those represented by Pinatubo-2 sample. Thus, the former may perform better as embankment materials. Pinatubo-2 sample was also found to be very compressible and may undergo large settlements when utilized as geotechnical structures. It is recommended that caution should be employed in their use and that the possibility of implementing appropriate soil improvement techniques should be pursued.

Based on the obtained stress-strain relations, the shear behavior of lahar can be incorporated into appropriate constitutive models. Zapanta [11] showed that the two-parameter hyperbolic model can represent accurately the experimental stress-strain relations and applied this model using a non-linear finite element analysis to numerically assess the performance of lahar as fill materials for embankments. However, the model can not account for the contractive and dilative response of the material. It is, therefore, recommended that appropriate stress-dilatancy relations be employed when incorporating volumetric change behavior in the modeling process.

Undrained torsional shear tests conducted to investigate the cyclic deformation characteristics showed that the relations between normalized shear modulus, damping ratio and shear strain of both samples were similar to those of granular materials. On the other hand, the liquefaction test results indicated that Pinatubo 2 sample, although relatively dense, had weak resistance against liquefaction while Pinatubo-1 sample in looser condition exhibited stronger resistance.

It is worth mentioning that some observations made in this research are not consistent with the findings in the earlier study conducted by Zarco et al. [6]. This emphasizes the differences in lahar properties with sampling sites. Thus, there is a need to establish a database of lahar properties. It is believed that the results of tests presented herein are supplementary information that could

contribute to the formulation of appropriate design methodologies on the use as of re-deposited lahar as construction materials, whether for embankments or for foundations.

VIII. Concluding Remarks

The geotechnical characteristics of re-deposited lahars taken from Mt. Pinatubo were investigated using a hollow torsional shear apparatus and the results were analyzed with the objective of examining their behavior when used as materials for foundations and embankments. Most of the geotechnical properties of the lahar samples examined, such as index properties, permeability characteristics, stress-strain relations, volumetric change response and dynamic deformation behavior, are similar to those of typical granular materials.

The contrasting results of the tests on two samples indicate that generally, lahar deposits at different sites can have quite different over-all geotechnical characteristics. It is believed that these variances can be attributed to the mode of deposition and its location or distance from the main source, i.e. Mt. Pinatubo volcano. Hence, care must be taken in the selection of re-deposited lahar which will be used as construction materials.

Since all the tests performed herein made use of reconstituted samples, the results are applicable to lahar deposits used as embankment materials. However, the effects of various factors, such as lateral stress, fabric, ageing and stress and strain history among others, are also important when investigating their behavior as foundation ground. Therefore, further laboratory tests using undisturbed samples and in-situ tests, such as plate loading tests, pressiometer tests, geophysical tests among others, are highly recommended.

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NOMENCLATURE

c	– cohesion (kPa)
D_{10}	– diameter corresponding to 10% passing (mm)
D	– damping ratio
Dr	– relative density (%)
e_{max}	– maximum void ratio
e_{min}	– minimum void ratio
Fc	– fines content (%)
G	– shear modulus (MPa)
G_0	– initial shear modulus (MPa)
G_s	– specific gravity
k	– coefficient of permeability (cm/sec)
h	– height of torsional specimen (mm)
p'	– mean effective confining pressure (kPa)
r_i	– inner radius of hollow torsional specimen (mm)
r_o	– outer radius of hollow torsional specimen (mm)
w	– water content (%)
γ_{at}	– shear strain
ε_v	– volumetric strain
ϕ	– angle of internal friction (degree)
σ'_z	– axial stress on the hollow torsional specimen (kPa)
σ'_r	– radial stress on the hollow torsional specimen (kPa)
σ'_θ	– circumferential stress on the hollow torsional specimen (kPa)
σ'_c	– initial effective confining pressure in liquefaction tests (kPa)
ρ_d	– dry density (g/cm ³)
τ	– torsional shear stress (kPa)
τ_{max}	– maximum cyclic torsional shear stress in undrained test (kPa)