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Geotechnical characteristics of volcanic soil from seismically-induced Aratozawa landslide, Japan.

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Abstract

This paper seeks to investigate the properties of volcanic soil from the Aratozawa landslide, the largest failure triggered by the 2008 Iwate-Miyagi Nairiku earthquake, Japan. The Aratozawa landslide, which extended about 1200 m long and 800 m wide, consisted of a complex system of ridges and depressions, including a number of smaller failures that occurred within the slide body. Field investigation was carried out to study the geotechnical properties of pumice that was exposed at the scarp of two smaller slides. The pumice was found to be heavily weathered, having low dry density and high moisture content. Portable cone penetration tests were performed to evaluate the in-situ properties of soil as well as to determine the thickness of the weathered zone. Laboratory examination included slake-durability tests, grain-size distribution analysis and a series of cyclic loading triaxial compression tests conducted on undisturbed and reconstituted samples. Laboratory test data indicated that the soil had a high potential for generation of excess pore-water pressures, suggesting that liquefaction might have occurred in the weathered mass of pumice during the earthquake.

Keywords Landslide, earthquake, mechanism, cyclic tests

Introduction

The Iwate-Miyagi Nairiku earthquake struck the Tohoku region of northeastern Honshu, Japan on June 14, 2008. The epicenter was located about 85 kilometers north of Sendai, and about 385 kilometers north-northeast of Tokyo (Fig. 1). The Japan Meteorological Agency estimated the magnitude (M_{JMA}) at 7.2, while the moment magnitude (Mw) measured by the USGS was 6.9. The earthquake triggered several slope failures, primarily deep-seated rotational slides in weathered rocks, as well as a few shallow translational slides and debris flows. According to Kayen et al. (2008), most of the large landslides that caused considerable damage to highways and bridges were concentrated within an area with a radius of 15-20 kilometers from the epicenter.

The Aratozawa landslide located in the vicinity of the Aratozawa Dam was clearly the largest failure caused by the earthquake (Fig. 2). The landslide extended about 1.2 kilometers long and 800 meters wide. The total volume of the sliding body was estimated to be 67 million m^3 (Miyagi et al. 2008). The height of a newly-created cliff (Fig. 3) reached up to 150 meters, making this landslide one of the largest failures to occur in the past few decades in Japan. Another peculiar feature distinguishing this landslide was extensive lateral movements of the soil mass. The material from the original bluff line moved laterally about 300 meters, carrying a 3.5 km-long section of road down slope.

A research group from the University of Tokyo conducted a field survey to evaluate the in-situ properties of soil involved in sliding and collected the data necessary to understand the mechanism of slide initiation and development. The survey showed that the shape of the landslide was significantly affected by displacements of the original ground surface, resulting in a complex system of ridges and depressions. The ridges were formed from relatively hard volcanic rocks while the depressions were mostly filled with fine-grained material (Fig. 4). Observations indicated that the ridges of volcanic rocks had undergone large displacements with almost no change in elevation, suggesting that the sliding occurred along a very gentle slip surface. Boring data obtained and published online by the Tohoku Regional Forest Office seem to lend support to this hypothesis. The Tohoku Regional Forest Office’s report (2008) indicates that the sliding surface developed at a depth of 150-200 m in a layer of siltstone sloping downward to the south by 3-4 degrees.

The Aratozawa landslide also produced about 4.0 million m^3 of debris material (Fig. 5). Miyagi et al. (2008) reported that within 10 minutes after the earthquake event, the toe section of the landslide broke into smaller blocks and plunged into the reservoir, causing a wave that overtopped the spillway of Aratozawa dam. Several slides of a smaller scale were also triggered on the ridges of volcanic rocks, forming slopes with steep inclinations of 40-50 degrees. The material exposed at the scarp of those slides was identified mostly as heavily weathered pumice. To shed light on the mechanisms of such failures, field investigation and laboratory examination of soil samples were conducted at the Geotechnical Laboratory of Tokyo University. During the field investigation, portable cone penetration tests were performed to evaluate the strength characteristics of the soil and the degree of weathering. Laboratory work included a series of undrained cyclic loading triaxial compression tests carried out to determine the dynamic properties of pumice.

This paper briefly describes the conditions in the area that contributed to slope instability, discusses the basic causes and mechanisms of the Aratozawa landslide, and reports the results of field and laboratory investigation.

Geology of the study area

The Aratozawa landslide area is located within a region where geological features as well as lithological units are primarily determined by previous volcanic activities. Fuhara et al. (2008) pointed out that the geology of the area is strongly influenced by the presence of a caldera-like structure. The caldera's floor is filled with relatively soft volcanic ash and breccia which are overlain by sedimentary debris resulted from erosion of adjacent volcanic highlands and volcanic deposits. The top part of the caldera is occupied with lavas and tuffs of Pleistocene erupted from postcollapse caldera-related vents. Yamada et al. (1986) reported that due to the previous volcanic activities, the study area contains a number of hot springs, leading to the formation of groundwater with low pH values (pH≈3.0). It is believed that such highly acidic groundwater has significantly precipitated the process of chemical weathering in the study area, making volcanic rocks rather loose and weakened (Fig. 6).

It is interesting to note that most of landslides triggered by the earthquake were located within the caldera structure (Fuhara et al. 2008). Kamiyama (2009) suggested two possible causes of these phenomena: 1) the caldera-like structure seems to have amplified the ground motion of the rocks, and 2) the great depth of the weak siltstone layer gave the material in this area a longer period of vibration.

Site conditions in the Aratozawa landslide area

The general soil conditions determined on the basis of boring data can be found in Fig. 7, where a soil profile along the slide zone of the Aratozawa landslide is presented (line

A-A’). As can be seen in Fig. 7b, the north part of the area where no sliding occurred is covered by a massive layer of volcanic rocks such as welded tuff and pumice. The volcanic rocks are underlain by a deep bed of sedimentary rocks, including siltstone with alternating layers of sandstone. Analysis of geomorphic features of the Aratozawa landslide area has prompted Chiba et al. (2008) to assume that the Aratozawa landslide was a partial reactivation of an existing old slide. Results of field investigation conducted in the Aratozawa landslide area in 1992 by Chiba et al. (2008) and in 2008 by the Tohoku Regional Forest Office (2008) seem to provide a basis for this hypothesis. Boring data combined with the results of SPT tests suggested that the slip plane had probably existed in a weak layer of siltstone in the south part of the Aratozawa landslide. The strength of this material was characterized by SPT N-values of about 5-7. Results of ring-shear tests reported by Hasegawa et al. (2009) also indicated that the strength of siltstone from the slip plane was low ($\phi' \approx 10°$, and $c \approx 0$ kPa), a finding that was primarily attributed to the presence of smectite.

Mechanism of the Aratozawa landslide

As previously noted, one of the prime factors leading to the large lateral displacements was the presence of a weakened zone formed in the siltstone layer. However the mechanics of soil movement above this zone were complex, involving the lateral displacement of old landslide mass and the newly-formed ridges of volcanic rock. Utilizing the field and laboratory data discussed above as well as the results of computer analysis performed by Hamasaki et al. (2009), the mechanism of the Aratozawa landslide can be described as follows. It is believed that a weakened zone was formed in the volcanic rocks, probably, in the early stages of the earthquake (Fig. 8b). Possible causes of the development of such a weakened zone were a loss of strength in the volcanic rocks due to the cyclic shear stresses and strains induced in them by the earthquake. Then, sliding of a mass initiated after the formation of failure plane in the volcanic material would likely occur along the already existing slip plane in the siltstone (Fig. 8c). Furthermore, as the earthquake shaking continued, deformations of the sliding body would probably cause a further loss of shear strength as the result of remolding in the shear zones, thus facilitating lateral displacement. Following the lateral translation of the newly-formed ridges, extensive tension cracking might have developed in the volcanic material behind the ridge. Settlement of the area behind the ridge formed a new scarp at the back of the slide area and a graben type of depression (Fig. 8d).

From the aforementioned analysis, it can be inferred that the slip plane, which had probably existed in the siltstone before the earthquake, significantly contributed to the development of the Aratozawa landslide, providing a lubricated surface on which blocks of soil from the sliding mass could move with little resistance to motion. Yet, little is known regarding the role of volcanic soils in the initiation of sliding. This paper reports and discusses the geotechnical characteristics of volcanic material obtained from field and laboratory investigations, thus shedding further light on the mechanism of the Aratozawa landslide.

Results of Field Investigation

Two sites where volcanic soil identified as pumice was exposed on the newly-created sliding surface were selected for detailed examination. The first point marked as P1 in Fig. 7a was located on the scarp of a relatively large slide near the toe part of the

Aratozawa landslide. The latter was located on the newly-formed ridge in a place denoted as P2 in Fig. 7a.

Portable cone penetration tests (Fig 9a) were conducted at P1 to evaluate the in-situ strength of pumice as well as the thickness of weathered mass. Results presented in Fig 9b indicate that the weathered zone extended to 1.5-2.0 m below the sliding surface. A few undisturbed soil samples were collected at both sites at the depths of 10-20 cm for laboratory investigation. Results of laboratory tests, which are summarized in Table 1, show that the pumice was in a relatively loose state, having a dry density in the range of 1.0-1.1, and the moisture content varying from 40-50%.

Results of Laboratory Investigation

Grain-size distribution analysis and slake-durability tests were carried out to determine the geotechnical properties of volcanic material. Grain-size distribution curve given in Fig. 10 indicates that the weathered volcanic material contained a relatively high amount of fine particles (≈23%). Slake-durability tests conducted in accordance with the procedure introduced by Franklin and Chandra (1972) yielded a relatively small slake-durability index of 20%, suggesting a high degree of soil deterioration.

Cyclic loading triaxial compression tests.

The dynamic properties of pumice were evaluated on the basis of cyclic undrained triaxial compression tests (JGS 0541-2000). Undisturbed soil samples collected from the sliding zone at P2 and specimens reconstituted in the laboratory were used. To preserve the integrity of undisturbed samples during specimen preparation, the following procedure was employed. The soil was first frozen, carefully retrieved from the sampler, and placed in a conventional triaxial apparatus. The frozen specimen was then isotropically consolidated and allowed to thaw at a room temperature. To minimize the disturbance of soil structure that can occur during the thawing process, confining pressures ($\sigma'_3 \approx 20$kPa) greater than the in-situ overburden pressure at the time of sampling were used (Yoshimi et al. 1978). Kiyota et al. (2009) noted that such a technique produces laboratory specimens with a limited degree of structure disturbance and thus changes in soil structure caused by the freeze-thaw process would have negligible effects on the properties of soil. After the triaxial specimen thawed, saturation was performed by purging the specimen with carbon dioxide for approximately one hour. Deaired water was then introduced into the specimen from the bottom drain line. The reconstituted specimens were produced by the air-pluviation method (JGS 0520-2000). The initial height and diameter of the test specimens prior to saturation were 75 and 150 mm, respectively. The specimens were prepared to have an initial dry density of 1.0 g/cm^3, a value that correlated to the dry density of the undisturbed samples (Table 1). A minimum B-value of 0.97 was obtained for all tests.

All the specimens were anisotropically consolidated using a principal stress ratio K_c, $K_c=\sigma'_1/\sigma'_3$, to account for the effect of a static shear stress on the strength of a soil element from the sliding surface. Although more detailed studies require laboratory test data to be obtained for different values of K_c (Seed et al. 1975), in this work, the single value of K_c=2.0 was used mostly due to the limited amount of testing material. However, it should be noted that K_c may influence the strength of soil as well as its resistance to liquefaction. For example, Ladd (1991) showed that the undrained strength of clay generally increases when it is consolidated under higher static shear stresses while Seed et al (1975) noted that soils consolidated to greater values of K_c generally require higher cyclic shear stresses to trigger failure.

Following complete consolidation, cyclic loading was applied under undrained conditions until the sample failed. Failure or liquefaction was defined in terms of a certain specified strain, that is 5% axial strain, following Lee and Roth (1977), and Ishihara (1996). The tests were performed with different cyclic stress ratios (CSR), and the number of cycles, N, of uniform stress required to cause failure was noted.

The results of two representative tests with undisturbed (a,b) and reconstituted (c,d) specimens initially consolidated to the same effective minor principal stresses of σ_{3c}'=50 kPa are shown in Fig.11. The effective stress paths are presented on the *p*'-*q* diagram, where *p*', effective mean normal stress, $p' = (\sigma'_1 + 2*\sigma'_3)/3$ is assigned to the horizontal axis and *q* ($=\sigma_1 - \sigma_3$) is plotted on the vertical axis. The same cyclic stress ratio (CSR=0.25) was used in both tests to estimate the effect of soil structure on the soil resistance to liquefaction. It is evident from Fig. 11b and Fig. 11d, that excess pore-water pressures generated during cyclic loading in both tests, leading to a loss in the effective mean normal stress. The samples responded with increasing strains with each cycle (Figs. 11a,c), but in no case was there a complete collapse typical of the classical liquefaction observed with loose clean sands. However, despite the similar patterns of cyclic behavior, the undisturbed specimen exhibited higher resistance to liquefaction as it required a greater number of cycles to failure. As can be seen in Fig. 11a, the specimen reached an axial strain of 5% in eighteen cycles of loading while the same amount of strain was induced in the reconstituted specimen only after seven cycles. Summary of all the tests performed with σ_{3c}'=50 kPa are presented in Fig. 12 as the number of cycles required to cause 5% axial strain against cyclic stress ratio (CSR).

It is noted that similar findings were reported by Yoshimi et al. (1984) who carried out triaxial tests on sand from Niigata, Japan. Yoshimi et al (1984) noted that, compared to the undisturbed (frozen) samples, the cyclic strength of reconstituted specimens significantly decreased as a result of structure disturbance.

Evaluation of liquefaction potential of pumice.

The possibility of liquefaction occurring in the pumice during the earthquake may be assessed by comparing the magnitude of cyclic shear stresses induced by earthquake ground motions with the cyclic stresses required to trigger liquefaction in cyclic loading tests for comparable initial conditions. Cyclic stress ratio (CSR) was estimated as part of ‘simplified procedure’ introduced by Seed and Idriss (1971), using the following expression:

$$CSR = 0.65\left(\frac{\sigma_{v0} a_{\max}}{\sigma'_{v0}}\right) r_d$$

in which a_{max} is the maximum horizontal acceleration at the ground surface in g’s, σ_{vo} and σ_{vo}’ are the total and effective vertical stresses at depth z. The parameter r_d is a stress reduction coefficient that accounts for the flexibility of the soil column.

Tawara (2009) reported that a maximum acceleration of 525gal was recorded during the earthquake by a seismograph installed inside the Aratozawa dam foundation. Unfortunately, little is known regarding the ground water conditions prior to the earthquake. Without more detailed investigation, which is out of scope of this research, it is not possible to determine whether the ground water table existed above the failure plane or the water in the landslide mass was distributed sporadically. Thus, for convenience of this analysis, it is assumed that the total stress was somewhat similar with the effective stress along the potential failure plane. The stress reduction coefficient of 0.9 was selected on the basis of empirical correlations between depth and the earthquake magnitude (Idriss 1999), resulting in the cyclic stress ratio (CSR) of about 0.3. According to Seed and Idriss (1982), the irregular earthquake loading produced by the 2008 Iwate-Miyagi Nairiku earthquake could be approximately represented by ten uniform load cycles.

Comparisons between the induced and required stress cycles are shown in Fig. 12, from which it may be seen that the computed value of shear stress (CSR≈0.3) developed in the soil would cause liquefaction in about six cycles. This finding suggests that the shear stresses induced by the 2008 Iwate-Miyagi Nairiku earthquake were sufficient to trigger failure in the weathered pumice.

Post-cyclic strength of pumice.

After the application of cycle loads, static compression tests were performed in order to obtain an indication of the strength remaining after the simulated earthquake loading. Without any additional consolidation, axial stress was increased until the samples developed large magnitudes of axial strain (Figs. 11a,c).

To study the effect of excess pore-water pressures on the post-cyclic strength of pumice, the obtained results are plotted in Fig. 13 in terms of effective and total stresses at failure. As can be seen in Fig. 13a, the effective strength characteristics of soil can be described as follows: $\phi' \approx 38.3°$ and $c' \approx 2$ kPa. However, when the test data are analyzed in terms of total stress conditions, the strength parameters are significantly low, measured to be: $\phi \approx 28.9°$ and $c \approx 4$ kPa (Fig. 13b). It is noted that the strength of undisturbed samples was found to be higher than that of reconstituted samples (Fig. 13).

Keeping in mind that effective stress conditions prevail in a slope before an earthquake while the total stress conditions are expected to exist immediately after the earthquake event, it may be assumed that some loss of strength would occur in the pumice due to the build-up of excess pore-water pressures. Results of undrained ring-shear tests conducted by Fukuoka et al. (2009) indicate that the pumice from the Aratozawa landslide can experience even a greater decrease in the strength ($\phi \approx 17.2°$) when subjected to large shear displacements of several meters.

Summary and Conclusions

The results of field and laboratory investigation into the geotechnical properties of pumice from the Aratozawa landslides that occurred during the Iwate-Miyagi Nairiku Earthquake of June 14, 2008 have been presented herein. The conclusions from the study are:

- The pumice was heavily weathered, having high values of moisture content.
- The slake-durability index was measured to be as low as 20%, suggesting a high degree of deterioration of the soil.
- The data from undrained cyclic loading triaxial compression tests revealed that the pumice had a high potential for generation of excess pore-water pressures during earthquake loading.
- Undisturbed soil samples of pumice exhibited greater resistance to liquefaction and strength than the specimens reconstituted in the laboratory.

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Figure captions

Figure 1. A map of Japan showing the epicenter of the 2008 Iwate-Miyagi Nairiku earthquake.

Figure 2. An aerial view of the Aratozawa landslide. Photo courtesy of the Asia Air Survey Co. Ltd.

Figure 3. A view of the cliff created by the landslide.

Figure 4. A part of the Aratozawa landslide, including ridges formed from volcanic rocks and depression.

Figure 5. Debris material at the toe of the Aratozawa landslide.

Figure 6. Images of weathered pumice: a) undisturbed soil sample used in a triaxial test; and b) the magnified (x100) surface of weathered pumice with secondary minerals of different color developed on it.

Figure 7. Plan of slide area (a), and soil profile through south end of slide area (b). After Tohoku Regional Forest Office (2008).

Figure 8. Hypothesized mechanism of slide development and movement. After Tohoku Regional Forest Office (2008), and Hamasaki et al. (2009).

Figure 9. Portable cone penetrometer used to evaluate the in-situ properties of volcanic material from the sliding zone (a); and results of portable cone penetration test (b).

Figure 10. Grain-size distribution curve of volcanic material used in triaxial compression tests.

Figure 11. Results of undrained cyclic loading triaxial compression tests plotted as: (a, c) deviator stress versus axial strain; and (b, d) deviator stress versus mean effective confining stress.

Figure 12. Summary of undrained cyclic loading triaxial compression tests.

Figure 13. Post-cyclic strength of the volcanic soil presented in (a) effective and (b) total stress conditions at failure (ϕ - friction angle, c –cohesion).