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In-situ cyclic direct shear tests on volcanic soil in the site of landslide due to earthquake

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ABSTRACT

Volcanic pumice soils are widely distributed in many countries. Mechanical properties of those need to be characterized since seismic ground motions can cause serious hazards such as large-scale slope failures and long-distance debris flows. Since volcanic pumice generally retains extremely high porosity and has a sensitive structure, the natural soil structure is easily broken during sampling or transportation. Therefore, it is preferable to conduct in-situ tests without disturbing the original structure of the soil. In this study, an in-situ direct shear test device was developed, and in-situ cyclic direct shear tests were performed on volcanic pumice (Ta-d). This pumice soil is considered to be the main cause of slope failure in Atsuma, Hokkaido, Japan, due to Easten Iburi earthquake in 2018. Laboratory direct shear tests were also conducted using undisturbed sample taken from the site. The results revealed that soil structure significantly affects shear strength. To evaluate the mechanical behavior of such sensitive soils, it is crucial to use specimens with as less disturbed specimens as possible. The newly developed in-situ direct shear test device was used to determine the cyclic shear strength of sensitive volcanic pumice soils.

Keywords: cyclic direct shear test, In-situ testing, volcanic pumice soils

1. Introduction

Volcanic pumice soils are found worldwide and have been known to cause large-scale slope failures (Bommer et al., 2002). For instance, in El Salvador, Central America, a major landslide occurred, likely due to volcanic pumice soils (Bommer and Rodriguez 2001). The Canary Islands in Spain are also composed of volcanic pumice soils, and past slope failures have been documented (Masson et al. 2001). However, the characteristics of volcanic pumice soils are not yet fully understood. Volcanic pumice soils are typically made up of delicate, porous particles and have a loose structure. The structure of these soils can be easily damaged during sampling or transportation, making it difficult to replicate the original structure. Therefore, it is better to conduct tests with the original intact soil structure.

This study presents the development of a lightweight and compact cyclic direct shear test apparatus. The apparatus enables testing to be carried out in mountainous areas where access is difficult. In-situ testing was conducted at a site that had collapsed due to earthquake. The in-situ test results were compared with laboratory tests using intact and reconstructed specimens to evaluate the effectiveness of in-situ testing in volcanic pumice soils.

2. In-situ direct shear test device

2.1. Components

Fig. 1A displays a photograph of the in-situ cyclic direct shear test apparatus, which is a modified version of the in-situ shear test apparatus developed by Hashimoto et al. 2023. This apparatus can replicate the upper movable and lower fixed version of the indoor direct shear test apparatus.

The apparatus comprises a rigid frame that houses the specimen, a top lid, a jack for applying shear stress, a displacement transducer, a load cell, and a microcomputer unit. The frame is square with a base of 120 mm per side and is designed to enclose a specimen of 60 mm height. The fixture was 3D printed to simplify construction and reduce its weight. The device weighs around 7 kg and can be manually carried to the site of a slope failure in mountainous terrain. It is equipped with one displacement transducer and two load cells to measure horizontal displacement, horizontal load, and vertical load, respectively. The digital data is obtained and stored in real-time by a micro-controller unit. However, measuring vertical displacements is not possible due to the structure of the device.

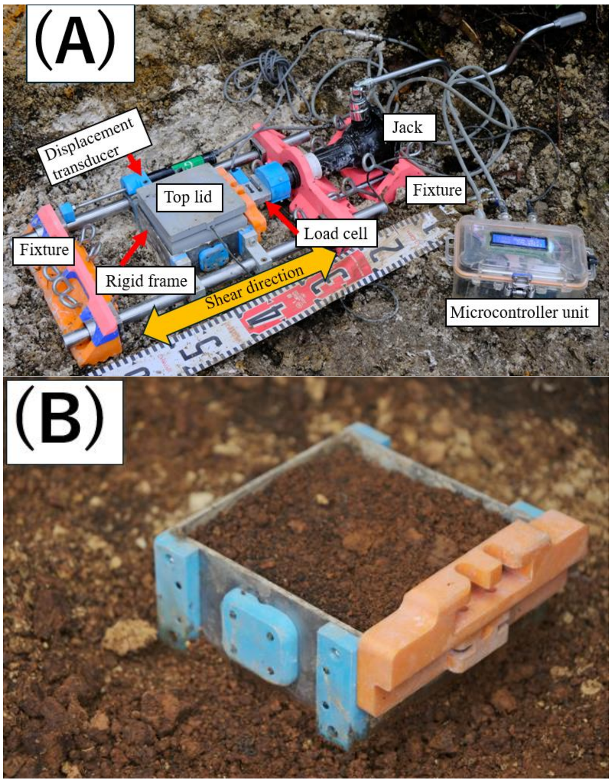

Figure 1. In-situ test apparatus, (A) Overhead view and (B) trimming and framing soil in a rigid frame.

2.2. Testing procedures

Fig. 2 shows the procedure for in situ direct shear tests. The slope was excavated horizontally, and a frame was placed on the trimmed specimen (Fig. 1B). The specimen was then covered with a lid (Fig. 2A). For soil specimens with a fragile or non-supporting structure, such as pumice soil, the specimen was prepared by gradually inserting the frame while removing the soil around it. The frame corresponds to the upper box of a direct shear test in a laboratory. The frame's horizontal movement creates a shear plane at the bottom of the specimen, specifically at the ground surface (Fig. 2C). The test can be conducted while preserving the soil structure around the shear plane.

Once the frame was installed, shafts were attached to it, along with front and rear fixtures, jacks, and various measuring devices (Fig. 2B). The fixture is secured to the ground by inserting anchor bolts into it. The shaft attached to the frame is then inserted into ball bearings attached to the fixture, allowing the shaft and frame to move simultaneously in shear.

Vertical stress can be applied by placing stones or a person on the top lid of the specimen. In direct shear tests, it is recommended to measure the vertical stress at the bottom of the lower box. However, in this device, the equivalent location of the bottom of the lower box is underground and cannot be measured with the original ground retained. Note that the vertical stresses near the shear surface may be smaller than those measured because of friction forces on the wall. Shear stresses are generated by screw-type jacks and can be adjusted by manually turning the handle. The direction of shear can be changed by the direction in which the handle is turned. The shear rate was set to be 10 mm/min, and digital data was recorded at 10 Hz cycles. The 5 V drive power supply for the measuring device and microcomputer unit was obtained by boosting an AA battery. Finally, the frame was removed to ensure that there were no large gravels on the fracture surface that could affect the test results.

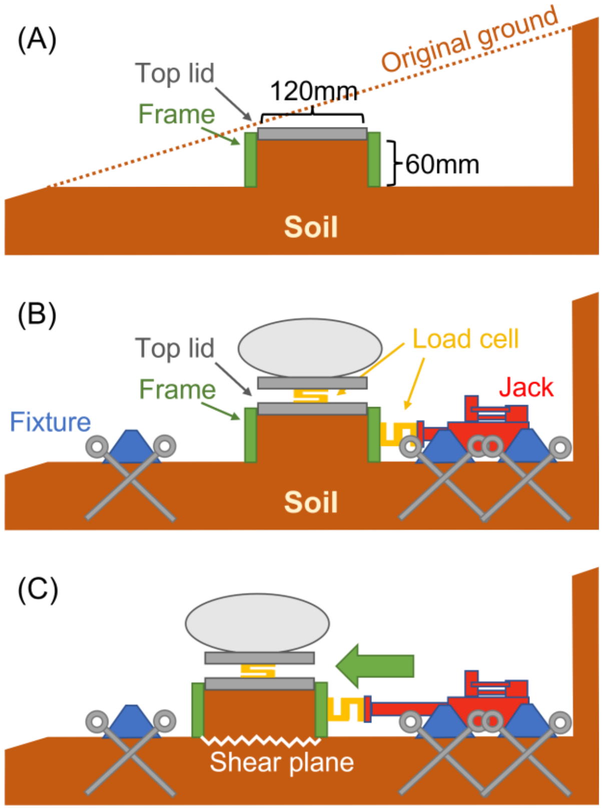

Figure 2. Testing procedures of the in-situ direct shear test. (A) Shear box installation, (B) loading and measuring devices installation and (C) shearing.

3. Slope failure in Atsuma, Hokkaido and its pumice soil

3.1. Slope failure in Atsuma, Hokkaido

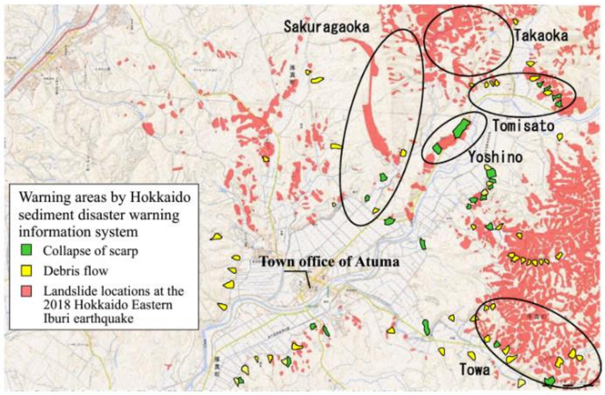

Figure 3. Landslide locations in Atsuma (Ishikawa, 2021)

On 6 September 2018, a 6.7 magnitude earthquake occurred in the central-eastern part of the Iburi region of Hokkaido, with an epicentre depth of 37 km (Ishikawa et al. 2021). The town of Atsuma, located near the epicentre, experienced strong seismic tremors that caused landslides and the discharge of volcanic deposits, resulting in the destruction of many houses. This event triggered numerous landslides, with an estimated total area of slope failure of 13.4 km^2, the largest in the last 150 years (MLIT 2018). Most of the landslides observed were shallow, with well-defined sliding surfaces and long sliding distances. The study focused on a selected area where the layers likely to have been caused by the collapse were moderately exposed, and where in-situ testing was easy to perform.

3.2. Tarumae d pumice (Ta-d pumice)

Fig. 4 shows reddish-brown pumice soil exposed in the investigated area. Based on its characteristics and the surrounding sedimentary layers, this pumice soil is Tarumae d (Ta-d), which was deposited approximately 9,000 years ago among the volcanic pyroclastic flow deposits of Tarumae Volcano. The Ta-d layer is considered to play a significant role in the slope failure caused by the earthquake. The Ta-d layer was thicker in the area where the slope failure occurred compared to the area where it did not occur (Kawamura et al. 2019).

Table 1 provides the physical properties of this pumice soil, which is characterized by a natural moisture content ratio higher than the liquid limit. This phenomenon is attributed to the retention of additional water in the intraparticle voids of the larger particles.

Figure 4. Ta-d pumice

Table 1. Physical properties of Ta-d pumice soil

ρ_s	ρ_d	W_n	e	D_{50} (mm)	W_P (%)	W_L (%)	I_p
2.68	0.30	219	7.93	2.5	151	206	55

ρ_s: Soil particle density, ρ_d: Dry density, w_n: Water content in natural condition, e: Void ratio, D_{50}: Mean particle diameter, w_p: Plastic limit, w_L: Liquid limit, I_p: Plasticity index

4. In-situ direct shear tests

4.1. Test conditions

Table 2 shows the results of in situ direct shear tests conducted on Ta-d. A vertical stress of 20 kPa was applied using a pre-prepared weight, and that of 35 kPa was applied by one person.

Table 2. Test conditions for in-situ direct shear tests

Case ID	Normal stress σ (kPa)	Normal load applied by	Shear rate (mm/min)
in-situ 20	20	Prepared weights	10
in-situ 35	35	One person	10

4.2. Results

Fig. 5 shows the relationship between shear stress (τ) and horizontal displacement (D) from in-situ direct shear tests. Peak strength was reached at around 13 mm. After reaching the peak, shear stress decreased in response to displacement but did not reach a full residual state, probably because the amount of displacement was not sufficient.

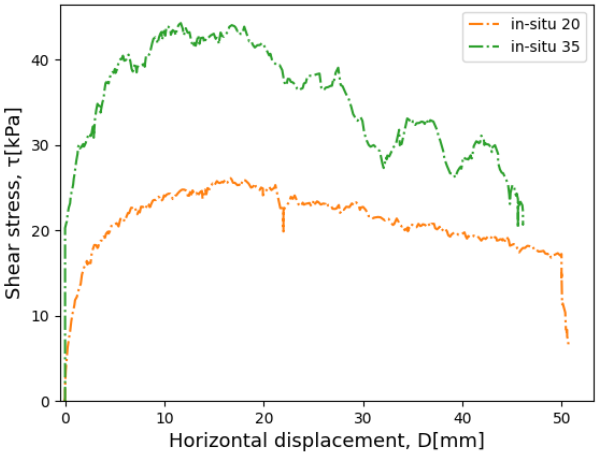

Figure 5. Results of in-situ direct shear tests

5. Laboratory direct shear tests

5.1. Test conditions

Direct shear tests were conducted on Ta-d under constant pressure conditions using a conventional direct shear test apparatus in laboratory. The specimens used were 80 mm square and 65 mm high. Tests were conducted on three intact specimens and three reconstructed specimens, as presented in Table 3. The intact specimens were transported with minimal soil disturbance by fitting them into a shear box created by a 3D printer. Intact specimens were trimmed on site and placed directly into shear boxes as shown in Fig. 6.

Figure 6. Field sampling of intact specimens

The reconstituted specimens were prepared using pumice soil with a natural moisture content. Although the water content of the reconstituted specimens was lower than that of the undisturbed specimens, it could not be adjusted as adding water would cause the soil to become muddy. This indicates the difficulty in reproducing the soil structure of the original ground with disturbed soil.

Table 3 shows the test conditions for the laboratory direct shear tests. The vertical stresses were set at 10, 20, and 35 kPa, which are comparable to those in situ. The consolidation time was determined using the 3t method, for approximately 1 hour. Shearing was conducted at 1 mm/min after 90 minutes of consolidation.

Table 3. Test conditions for laboratory direct shear tests

Case ID	Specimen type	w (%)	σ (kPa)	Shear rate (mm/min)
Intact 10	Intact	311	10	1
Intact 20		222	20	
Intact 43		217	43	
Re-con 10	Reconstituted (Re-con)	193	10	
Re-con 20		187	20	
Re-con 35		179	35	

5.2. Results

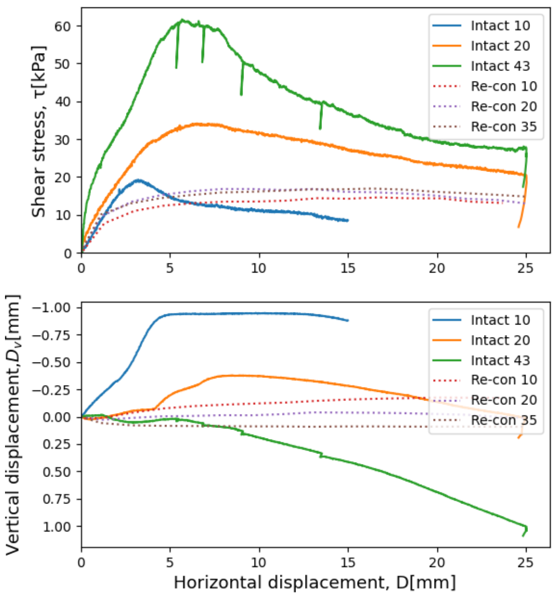

Figure 7. Results of laboratory direct shear tests (A) horizontal displacement and shear stress, (B) horizontal displacement and vertical displacement

Shear stresses (τ) and vertical displacements (D_v) of laboratory direct shear tests are plotted against horizontal displacements (D) in Fig. 7.

The peak strength of the intact specimens was at approximately 3 mm when subjected to σ = 10 kPa and at approximately 7 mm when subjected to σ = 20, 43 kPa.

None of the reconstructed specimens showed post-peak strain softening. The reconstructed specimens maintained a similar peak strength despite variations in vertical stress, which was significantly lower than that of the intact specimens at the corresponding normal stress.

Intact 43 exhibited significant contraction, while intact 10 and intact 20 showed dilation during shearing. The reconstructed specimens, in general, did not undergo any volume change.

6. Comparison of peak strength between in-situ and laboratory direct shear tests

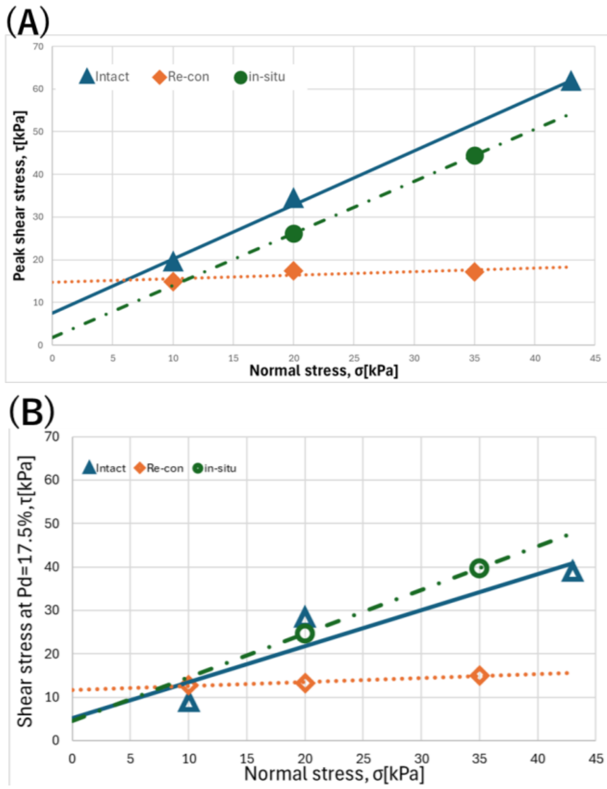

Figure 8. (A) Peak shear strength, (B) residual shear strength for in-situ and laboratory

Fig. 8A shows the peak shear stress (τ_{peak}) plotted against the normal stress (σ) from the in-situ and laboratory tests. The trends observed in the in-situ tests and the intact specimens in the laboratory tests are similar. Notably, the internal friction angle (φ_{peak}) for the in-situ tests was φ_{peak} = 51.7°, while for the intact specimens it was φ_{peak} = 50.6°, which is almost identical. The intact specimens exhibited higher peak stresses when compared at the same normal stress of in-situ cases, which may be due to the lower water content during the period between trimming and testing. The reconstructed specimens had a peak angle of internal friction (φ_{peak}) of 4.7°, which was significantly lower than that of the in-situ and intact specimens. Soil structure seemed to be completely destructed.

Fig. 8B illustrates the relationship between residual shear stress (τ_{res}) and vertical stress (σ). τ_{res} is defined as the shear stress at a relative displacement (P_d) of 17.5%, where relative displacement was defined to be percent ratio of horizontal displacement and shear box length. In other words, it is the shear stress at D = 21 mm for the in-situ test (120 mm square) and at 14 mm for the laboratory test (80 mm square). In both in-situ and laboratory tests, the residual strength was found to be lower than the peak strength. However, a drop of the shear stress observed in the reconstructed specimens were minimal. It seemed that a larger displacement is required to obtain real residual strength.

7. Cyclic direct shear test for residual shear strength

Tests have been conducted on the residual strength of soils for a long time (Skempton 1964). The cyclic direct shear test has also been proposed as a method for measuring the residual strength of soils (Nakamori, Yang, and Sokobiki 1996). Repeated displacement creates large accumulated deformation of the soil. Note that the shear direction reverses sequentially, which may affect the slip surface conditions.

Suzuki (2007) proposed a method for determining residual strength from cyclic direct shear tests using the hyperbolic approximation. This method expresses the relationship between the stress ratio τ/σ_N as a hyperbolic curve.

$$\frac{\tau}{\sigma_N} = \frac{N}{a+bN} \quad (1)$$

Both 'a' and 'b' are experimental constants obtained by least-squares fitting to the measured values. If the measured values are well-fitted, the stress ratio in the residual state can be determined.

$$(\frac{\tau}{\sigma_N})_r = \frac{1}{b} \quad (2)$$

Cyclic direct shear tests were conducted both in-situ and in-laboratory following this method. In this study, it is considered that the residual state was not reached because sufficient displacement could not be applied due to the testing machine. Therefore, the “fold strength” was defined as the strength at relatively small repetitive displacements before the peak strength was reached.

8. In-situ cyclic direct shear test

8.1. Test conditions

Cyclic direct shear tests were conducted in-situ on Ta-d by applying a vertical stress (σ) of 35 kPa, as presented in Table 4. The stress was applied by a single individual. The repeated displacement given was ±ΔD=7 mm from the initial position.

Table 4. Test conditions for in-situ cyclic direct shear tests

σ (kPa)	Normal load applied by	Shear rate (mm/min)	ΔD	N Number of repetition
35	One person	10	7	3

8.2. Results

Fig. 9 illustrates the correlation between shear stress (τ) and cumulative horizontal displacement (ΣD) obtained from in-situ cyclic direct shear tests. Similar to the ordinal direct shear test, minor oscillations are visible due to the nature of the jacking. The strength at turning point (ΔD = 7 mm, P_d = 5.83%) decreased slightly with an increasing number of cycles, which is believed to be due to approaching the residual strength.

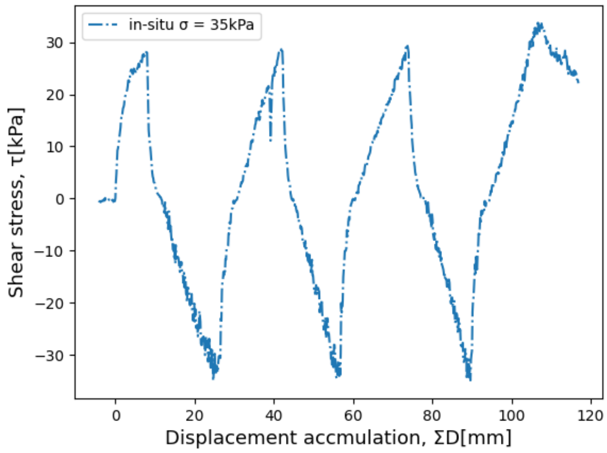

Figure 9. Results of in-situ cyclic direct shear tests

9. Laboratory cyclic direct shear tests

9.1. Test conditions

The laboratory cyclic direct shear tests were conducted with the intact and reconstructed specimens, as indicated in Table 5. The size of shear box used was different from the one used in-situ, so the displacement of ±ΔD=4.67 mm was applied to achieve the same relative displacement of 5.8% as in-situ tests. The shear displacement rate remained constant at 1 mm/min.

Table 5. Test conditions for laboratory cyclic direct shear tests

Case ID	Specimen type	w (%)	σ (kPa)	Shear rate (mm/min)	ΔD (mm)	N
Intact cyclic	Intact	145	35	1	4.67	3
Re-con cyclic	Re-con	240	35	1	4.67	2

9.2. Results

Fig. 10 illustrates the relation between shear stress (τ) and cumulative horizontal displacement (ΣD) obtained from laboratory cyclic direct shear tests.

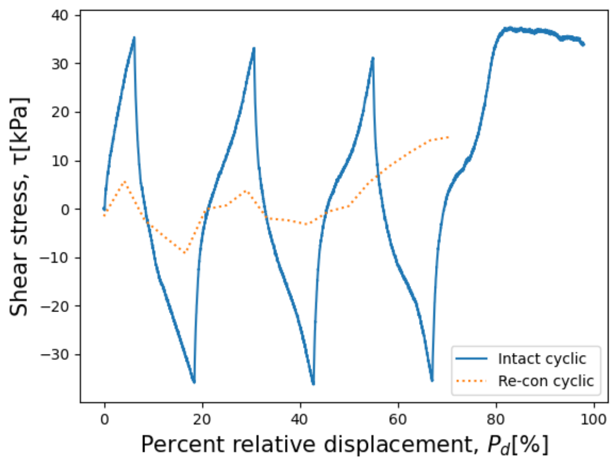

Figure 10. Results of laboratory cyclic direct shear tests

The shear strength at the fold point (ΔD = 4.7 mm, Pd = 5.8%) of both re-con and intact specimens decreases with the number of repetitions. As with the monotonic direct shear test, the shear strength of the intact specimens was found to be higher than that of the reconstructed specimens.

10. Comparison of residual strength between in-situ and laboratory cyclic direct shear tests

Fig. 11 shows the relationship between relative displacement and shear stress. For clarity, the intact line has been changed to a lighter colour; the intact and in-situ shear stresses were almost identical, with the re-con having significantly lower strength. Behaviour of intact and in-situ tests are almost the same, indicating the soil structure retained for both cases.

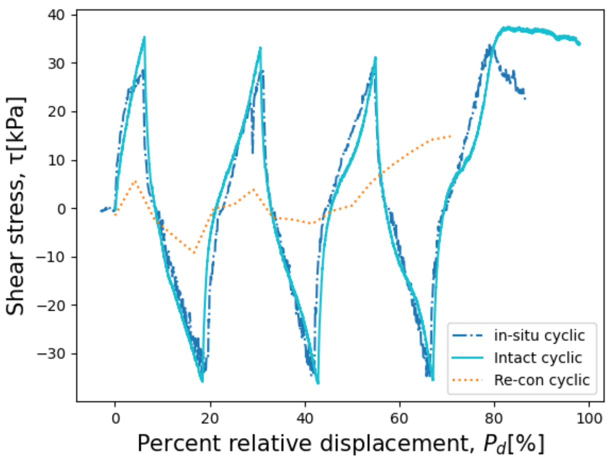

Figure 11. Relationship between shear stress and percent relative displacement

Fig. 12 shows the relationship between N/(τ/σN) and N (number of repetitions) for fitting the measured values from the in-situ and laboratory tests to Eq. 1. The slope of the straight line, determined using the least-squares method, corresponds to b in Eq. 1. The strong correlation between the measured values and Eq. 1, demonstrated by their linear relationship in all tests, suggests that Eq. 2 is a suitable model to use. The coefficient b was found to be 1.2 for the in-situ tests, 1.2 for the intact specimens, and 11 for the reconstructed specimens.

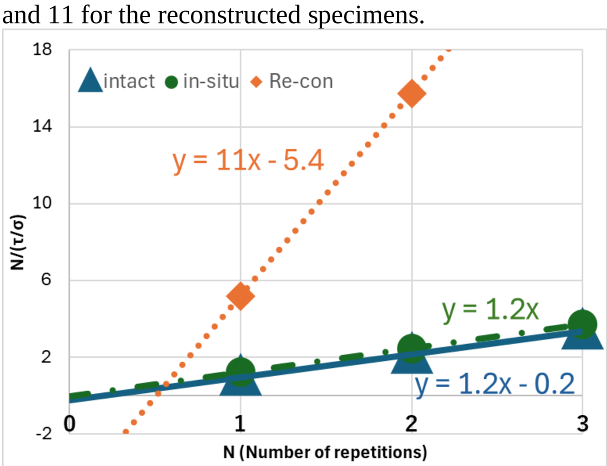

Figure 12. Relationship between N/(τ / σ N) and N

Fig. 13 illustrates the fold shear stress (τ_{fold}) obtained from the direct shear test plotted against the normal stress (σ), along with the relationship between fold strength and normal stress obtained from cyclic direct shear tests and Eq. 2. The relationship between fold strength and normal stress obtained from cyclic direct shear tests is added in Fig. 8. The shear resistance angles for the in-situ and intact tests were found to be almost the same, at 38.8° and 40.0° respectively. However, the reconstructed specimen had a very small shear resistance angle of 5.4°. The shear resistance angles were similar for all specimens when comparing the monotonic and cyclic direct shear tests.

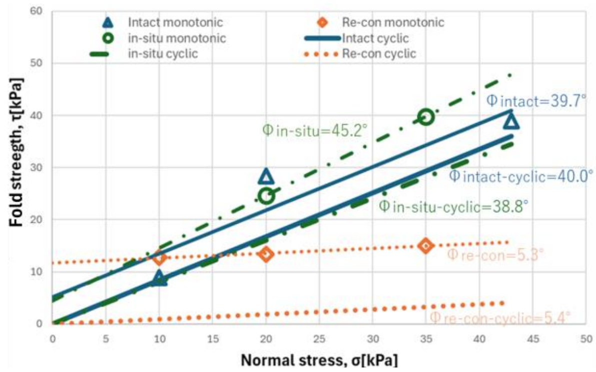

Figure 13. Residual shear strength for in-situ and laboratory cyclic direct shear tests

11. Conclusions

In this study, direct shear tests were conducted at a large slope failure site in Japan in 2018 to measure the parameters required for slope stability analysis. As the intact volcanic pumice in the target area is very fragile and its structure may change during transport, a lightweight and compact in-situ direct shear test apparatus was developed. A series of laboratory direct shear tests were also carried out on the same soil for comparison. Intact specimens were placed in shear boxes created by a 3D printer at the site.

The results of the in-situ and laboratory direct shear tests suggest the following with respect to Ta-d pumice.

a) In-situ tests and laboratory tests with undisturbed intact specimens exhibited similar behaviour, showing distinct peak stresses followed by strain softening., while reconstituted specimens had significantly low peak stresses. The preservation of soil structure seemed to be important to evaluate the strength of pumice soil.

b) It was found that the shear stress at the folding displacement decreased as the number of cycle increased. However, folding back before obtaining peak intensity did not lead to a residual state. The cyclic shear tests showed that the same behaviour was observed for both in-situ and intact specimens, provided that the fold displacements were the same.

In this study, we conducted repeated direct shear tests at relatively small displacement amplitudes and concluded that residual strength was not reached. We plan to carry out repeated tests at larger displacement amplitudes.

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