

Article

Earthquake-Induced Flow-Type Slope Failure in Weathered Volcanic Deposits—A Case Study: The 16 April 2016 Takanodai Landslide, Japan

Gabriele Chiaro ^{1,*} , Takashi Kiyota ², Muhammad Umar ³  and Claudio Cappellaro ¹ 

¹ Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch 8041, New Zealand

² Institute of Industrial Science, University of Tokyo, Tokyo 153-8505, Japan

³ Department of Civil Engineering, National University of Computer and Emerging Sciences, Lahore 54770, Punjab, Pakistan

* Correspondence: gabriele.chiaro@canterbury.ac.nz

Abstract: The aim of this paper is to provide new insight into the catastrophic mobility of the earthquake-induced flow-type Takanodai landslide that occurred on 16 April 2016, which had fatal consequences. A geological and geotechnical interpretation of the site conditions and experimental investigations of the mechanical behavior of weathered Kusasenrigahama (Kpfa) pumice are used to characterize the landslide failure mechanism. The results of large-strain undrained torsional shear tests indicate that Kpfa pumice has the potential to rapidly develop very large shear strains upon mobilization of its cyclic resistance. To evaluate the actual field performance, a series of new liquefaction triggering analyses are carried out. The liquefaction triggering analyses indicate that Kpfa pumice did not liquefy during the M_w 6.2 foreshock event, and the hillslope remained stable. Instead, it liquefied during the M_w 7.0 mainshock event, when the exceedance of the cyclic resistance of the Kpfa pumice deposit and subsequent flow-failure type of response can be considered the main cause of the landslide. Moreover, the combination of large cyclic stress ratios ($CSR = 0.21$ – 0.35)—significantly exceeding the cyclic resistance ratio $CRR = 0.09$ – 0.13)—and static shear stress ratios ($\alpha = 0.15$ – 0.25) were critical factors responsible for the observed flow-type landslide that traveled more than 0.6 km over a gentle sloping surface (6° – 10°).

Keywords: weathered volcanic soil; Takanodai landslide; flow-type failure; torsional shear tests; stability analyses; liquefaction triggering



Citation: Chiaro, G.; Kiyota, T.; Umar, M.; Cappellaro, C.

Earthquake-Induced Flow-Type Slope Failure in Weathered Volcanic Deposits—A Case Study: The 16 April 2016 Takanodai Landslide, Japan. *Geosciences* **2022**, *12*, 394.

<https://doi.org/10.3390/geosciences12110394>

geosciences12110394

Academic Editors:

Jesus Martinez-Frias

and Mohamed Shahin

Received: 4 October 2022

Accepted: 19 October 2022

Published: 25 October 2022

Publisher's Note: MDPI stays neutral with regard to jurisdictional claims in published maps and institutional affiliations.



Copyright: © 2022 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC BY) license (<https://creativecommons.org/licenses/by/4.0/>).

1. Introduction

On 16 April 2016, a relatively gentle slope (12° – 15°) located within the Mt. Aso Volcanic Caldera, in the Prefecture of Kumamoto (Kyushu Island, Japan), failed catastrophically due to severe ground shaking induced by the moment magnitude M_w 7.0 2016 Kumamoto Earthquake [1–4]. The resulting flow-type slide, which involved weathered volcanic soil deposits on a gentle slope, destroyed seven houses (causing five fatalities), damaged critical lifelines and local roads in the small residential Takanodai Housing Complex (Minamiaso Village), and threatened the neighboring Tokyu Country Town community.

Earthquake-induced flow-type landslides in volcanic deposits have occurred many times in the past [5,6]. Despite many research efforts, their mechanisms are still only poorly understood, making it difficult to predict in advance their occurrence and destructive consequences on the natural and built environment. Some of the major limitations in the evaluation of flow-type landslide hazard in similar settings are the lack of experimental data available on the cyclic and dynamic response of volcanic deposits, and the fundamental differences in behavior between such problematic soils and well-understood “textbook” soils (e.g., hard-grained quartz sands). The recent Takanodai flow-type landslide case

history emphasizes the current need to better understand the seismic response of volcanic soil deposits, and provides a critical opportunity to attain invaluable information toward such improved understanding.

Between April and October 2016, as a part of the Japan Society of Civil Engineers (JSCE) “Kumamoto Earthquake Damage Reconnaissance Mission”, the New Zealand Society for Earthquake Engineering “Learn from Earthquake—Kumamoto Mission”, and the “J-Rapid Japan-NZ Collaborative Kumamoto Project”, a research team involving the first two authors carried out post-earthquake damage reconnaissance surveys and geotechnical field investigations in the area of the Mt. Aso Caldera affected by the earthquake [3,4]. The research team carried out a comprehensive investigation program consisting of geological and geotechnical characterization of the Takanodai landslide site involving sampling of volcanic soils for subsequent geotechnical laboratory testing and static and time-history seismic slope stability analyses. The results of the slope stability analyses are presented in Chiaro et al. [7,8]. Preliminary analyses of laboratory data from torsional simple shear tests performed by the authors are given in Chiaro et al. [9] and Umar et al. [10].

Due to its engineering significance, the Takanodai landslide has also been studied by other investigators [2,11–13]. Other laboratory studies on Kpfa pumice, identified as the critical soil unit associated with the slope failure, include undrained cyclic and dynamic ring-shear tests of saturated specimens [2], undrained cyclic triaxial tests of both undisturbed and reconstituted saturated specimens [11], monotonic and cyclic direct shear box tests on both saturated and unsaturated [12], and constant vertical stress and constant volume cyclic direct shear tests [13]. Dang et al. [2] and Kasama et al. [12] used their experimental data to calibrate numerical models for the simulation of triggering and runout of the landslide.

This paper contributes to the abovementioned previous research efforts with a comparative analysis of experimental data from monotonic and cyclic torsional simple shear tests to highlight fundamental differences in the cyclic response of the Kpfa pumice at the Takanodai landslide site, as well as of hard-grained, liquefiable sands. Laboratory tests are carried out using a testing device capable of reaching very large shear strains to assess the potential of the tested soils to undergo flow-type failure upon triggering of liquefaction. Liquefaction triggering analyses are then carried out with an analytical procedure, accounting for the combined effects of earthquake-induced cyclic shear stress and driving static shear stress due to sloping ground conditions. Limitations on the applicability of the procedure to volcanic soils, and recommendations for future developments are then discussed.

2. The Takanodai Landslide: Characteristics and Field Observations

2.1. Geology of the Mt. Aso Volcanic Caldera

The landslide site is located nearby the Minami-Aso Township, within the Mt. Aso Caldera, on Kyushu Island in southern Japan (Figure 1). A series of active volcanic vents constitutes the inner caldera of Mt. Aso, the highest peak being Mt. Taka (1592 m above sea level), and it is encircled by the outer caldera, whose diameter varies between 18 and 25 km. To the west, the outer caldera is cut through by the Shirakawa River [4].

The rocks in the area of the landslide site, on the western side of the inner caldera, are mainly volcanic rocks of late Pleistocene age and consist of non-alkaline felsic and mafic volcanic rocks, comprising ignimbrite, volcanic breccia, and some basalt lava flows [14]. The lower slopes of the inner caldera have usually milder angles ($<10^\circ$) and consist of ignimbrites and lava flows overlain by a cover, which can be many meters thick, of volcanic deposits, including pumice. The upper slopes can instead be very steep ($>60^\circ$), and the overlying soil deposits tend to be shallower, with thicknesses of up to 10 m [4].

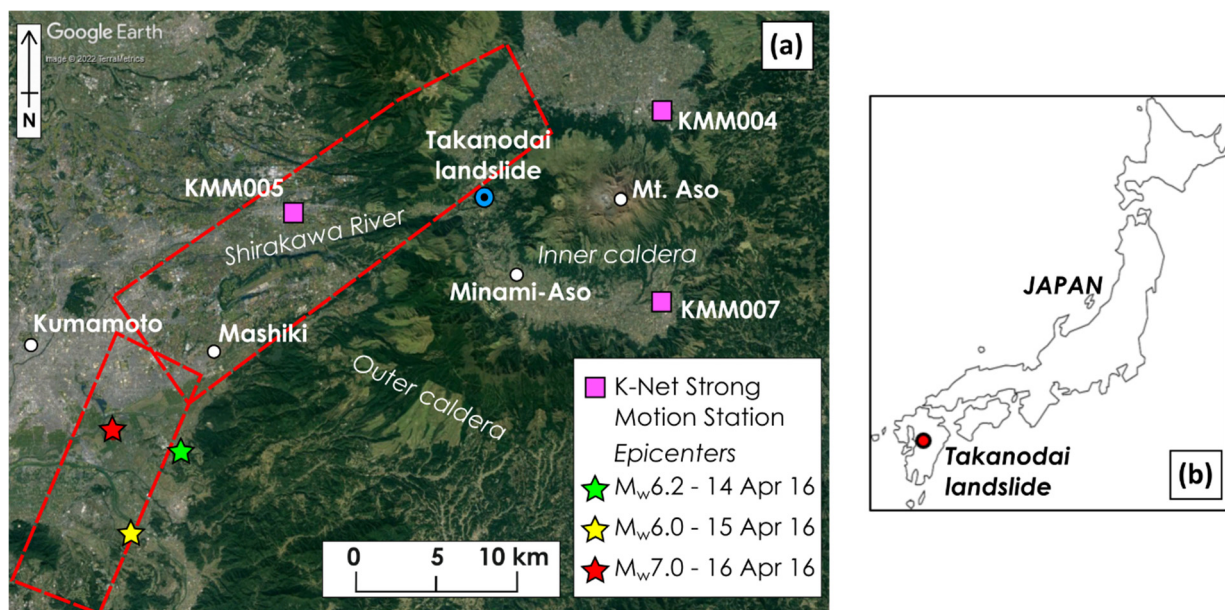


Figure 1. (a) Map of the epicentral area with location of landslide site, K-Net strong motion stations, and epicenters of main seismic events (base map from Google Earth; epicentral locations according to AIST [15]); the dashed red rectangles approximate the surface projection of the source model for the $M_w7.0$ event [16]. (b) Location of the landslide site in southern Japan.

2.2. 2016 Kumamoto Earthquake Sequence

In April 2016, a series of earthquakes affected the area of the Kumamoto prefecture, Japan. The seismic sequence comprised three events with moment magnitude M_w equal to or greater than 6.0 [1–4]:

- 14 April 2016 event (foreshock), $M_w6.2$;
- 15 April 2016 event (foreshock), $M_w6.0$;
- 16 April 2016 event (main shock), $M_w7.0$.

Epicentral locations for these seismic events are shown in Figure 1. The figure also shows the approximate surface projection of the finite source model for the $M_w7.0$ main shock as modeled by Asano and Iwata [16]; other studies on the source rupture process for this seismic event have adopted similar source models (e.g., [17,18]). The landslide site was located about 27 km from the epicenter of the main event, but its Joyner–Boore distance R_{JB} (i.e., the shortest distance between the site and surface projection of a three-dimensional rupture) was less than 2 km. The epicentral distances for the 14 April $M_w6.2$ and 15 April $M_w6.0$ events were about 25 and 30 km, respectively.

Figure 1 shows the location of three K-Net strong motion stations (KMM004, KMM005, and KMM007); all strong motion stations were located 12 km away from the landslide site. Stations KMM004 and KMM007 are underlain by 13–15 m of volcanic ash deposits over volcanic rock, while station KMM005 is underlain by 6 m of sand and 5 m of volcanic ash overlying volcanic rock (K-net [19]). Intensity measures and source-to-site distances for the ground motions recorded by these strong motion stations for the three largest earthquakes of the Kumamoto seismic sequence are summarized in Table 1.

Table 1. Intensity measures of recorded strong ground motions at K-Net stations for main events of Kumamoto earthquake sequence.

Date and M_w of Event	Station	R^+ (km)	PGA^{++} (g)	$S_a(1\text{ s})^{++}$ (g)	I_a^{++} (m/s)	D_{5-95}^{++} (s)	CAV^{++} (m/s)
14 April 2016, 6.2	KMM004	36	0.04	0.03	0.03	27.6	2.40
	KMM005	16	0.21	0.18	0.28	11.6	6.24
	KMM007	31	0.18	0.05	0.58	15.7	8.62
15 April 2016, 6.0	KMM004	42	0.01	0.02	0.00	41.2	1.00
	KMM005	22	0.05	0.05	0.03	12.8	1.72
	KMM007	35	0.08	0.02	0.10	16.2	3.40
16 April 2016, 7.0	KMM004	7	0.31	0.53	1.84	12.0	14.83
	KMM005	0	0.51	0.41	3.79	11.7	22.47
	KMM007	13	0.35	0.24	1.79	14.9	16.60

R = source-to-site distance (see note $^+$ below); PGA = peak ground acceleration; $S_a(1\text{ s})$ = spectral acceleration at 1 s period; I_a = Arias intensity; D_{5-95} = significant duration; CAV = cumulative absolute velocity. $^+$ For the 16 April $M_w 7.0$ event, the source-to-site distance was taken as the Joyner–Boore distance R_{JB} , while, for the other two events, the epicentral distance is reported. $^{++}$ Reported values of intensity measures are geometric mean values of EW and NS components of strong motion record.

2.3. Field Damage Observations at Takanodai Landslide Site

The Takanodai landslide took place on the gentler lower slopes of the inner caldera and involved the surface cover of volcanic deposits. These soils are known to be sensitive to pore-water pressure changes and earthquake loading, and there have been numerous past studies on rainfall-induced (e.g., [20]) and earthquake-induced (e.g., [6]) landslides in these materials.

As displayed in Figure 2a, the Takanodai landslide traveled in three different directions from a common source. The main slip (cross-section A–A', Figure 2a,b) had a length of about 600 m, a width of 100 m, and a thickness of 5–10 m. Field observations suggest that this was a shallow mobile earth slide. The presence of relatively large intact blocks of soil (Figure 3a,b) that traveled toward the slope toe on a 6° – 10° angle clearly indicates a translational movement of the soil mass, characterized by a flow-type failure. Numerous tension cracks above the head scarp and at the hilltop, adjacent to the Aso Volcanological Laboratory, were also observed (Figure 3c). Traces of an orange pumiceous soil were visible on the identified slip surface (Figure 3d), which consisted of a stiff clay-like ash deposit.

As schematically shown in Figure 4a, the soil profile at the Takanodai landslide site consists of an alternating sequence of volcanic deposits: (1) black volcanic ash with high organic matter (OL), (2) brown volcanic ash (AC), (3) weathered orange Kusenrigahama pumice (Kpfa), (4) stiff brown volcanic ash (Tp), and (5) soft/weathered volcanic rock. Borehole logs in the area indicate that the thickness of the Kpfa pumice layer varies between 0 and 1 m [4,11].

From the field observations, it was inferred that the brown volcanic ash deposits (AC and Tp), within which the Kpfa layer was sandwiched, are less permeable than the Kpfa pumice. The earthquakes were preceded by heavy rainfalls, and water seepage was evident within the pumice soil layer at the time of the first field survey in early May 2016 [4]. This suggests that the Kpfa pumice likely had a high degree of saturation at the time of the Kumamoto earthquakes [4,11].

Following the May 2016 field damage survey, it was hypothesized that the Kpfa pumice was the critical layer associated with the slope failure. Therefore, disturbed and undisturbed samples of the orange Kpfa pumice were retrieved for index testing and to evaluate the undrained response of this soil using torsional simple shear tests. Additional samples of the black (OC) and brown (AC and Tp) ash deposits were also collected to evaluate the unit weight of these deposits (Table 2).

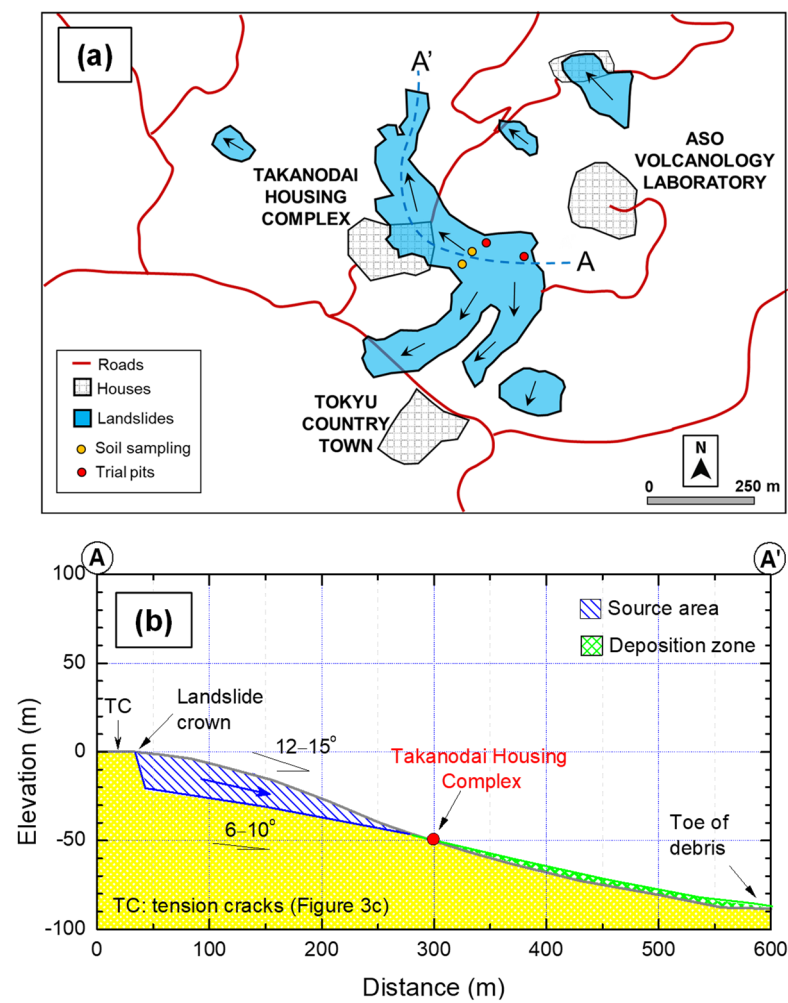


Figure 2. Takanodai landslide (N 32.8851; E 131.0049): (a) schematic plan view; (b) cross-section A–A' of the main slip.

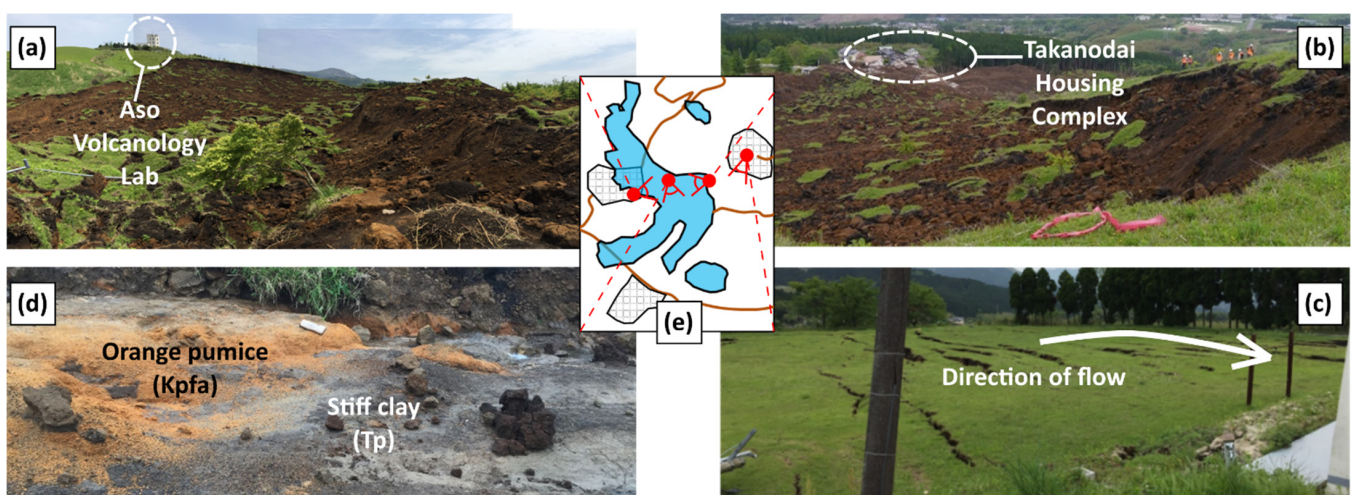


Figure 3. Photos of the Takanodai landslide site: (a) view looking uphill; (b) view looking downhill; (c) tension cracks observed at the hilltop; (d) identified slip surface on intact stiff clay (Tp) with remnants of Kpfa pumice layer. The inset (e) shows locations and direction of photos (a–d) with respect to the landslide; refer to Figure 2 for symbols used in map. Photos were taken in May 2016.

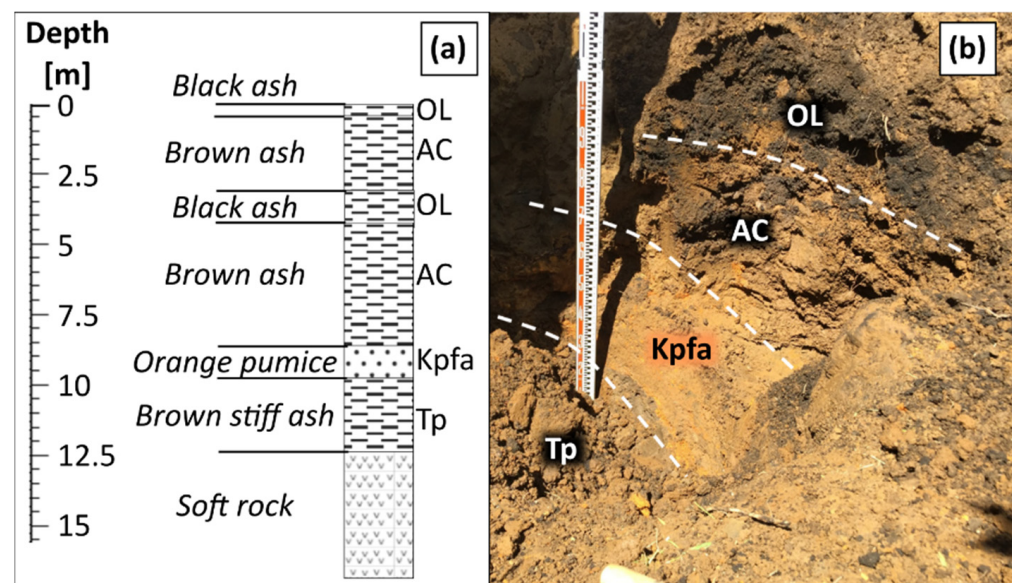


Figure 4. (a) Schematic soil profile and (b) matching volcanic soil deposits, as observed in wall of trial pit excavated on the northern side of the landslide (location shown in Figure 2); photo was taken in May 2016.

Table 2. Soil types identified at the Takanodai investigation site with measured in situ bulk unit weight.

Soil Type	Symbol [†]	Bulk Unit Weight (kN/m ³) ^{††}
Black ash	OL (1)	11.2
Brown ash	AC (1)	12.9
Black ash	OL (2)	11.2
Brown ash	AC (2)	19.2
Orange pumice	Kpfa	11.3
Stiff brown ash	Tp	19.2
Soft rock	—	22.0

As reported in [†] Hazarika et al. [11] and ^{††} Chiaro et al. [7].

3. Laboratory Testing of Kpfa Pumice

3.1. Test Material

Kpfa pumice samples were collected from the main slide of the Takanodai landslide (sampling locations shown in Figure 2a). The pumice had a specific gravity of 2.3 (average value) and in situ dry density of approximately 600 kg/m³; the in-situ water content at the time of sampling was 117%. Figure 4a compares its particle size distribution curve (orange curve) with that of the other soil layers OL, AC and Tp. A photo of an intact sample of Kpfa pumice overlaying brown stiff clay (Tp) is shown in Figure 5b, while scanning electron microscope (SEM) images are presented in Figure 5c,d. The Kpfa pumice is a pyroclastic soil that consists of silt and sand grains in a semi-continuous anisotropic matrix of halloysite material (Figure 5c,d). Kasama et al. [12] report that the particles of Kpfa pumice are less resistant than those of hard-grained silica sands (such as Toyoura sand), and they can be crushed with fingers.

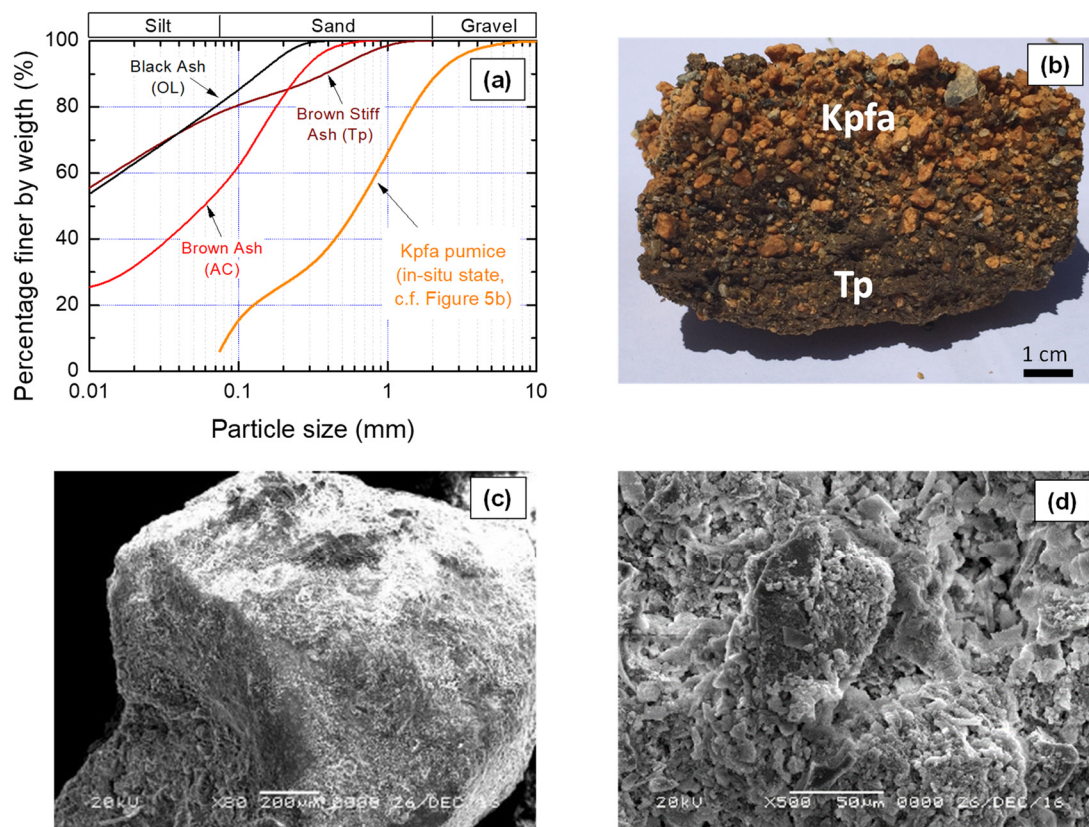


Figure 5. Volcanic soils investigated in this study: (a) particle size distribution curves (PSD for OL, AC and Tp are adopted from Kasama et al. [12]; (b) photo of an intact sample of Kpfa pumice overlaying Tp clay; (c,d) SEM images of Kpfa pumice.

3.2. Test Apparatus

A torsional shear apparatus on hollow cylindrical specimens is recognized to be a good tool to properly evaluate liquefaction soil response [21,22]. In particular, it offers the possibility to reproduce simple shear conditions that are a close representation of field stress conditions during earthquakes [23–26]. Moreover, in torsional shear tests on hollow cylindrical specimens, it is also currently possible to achieve higher strain levels by increasing the amount of torsional shear displacement that is applied to the soil specimen through the rotation of the top cap [21,27]. For instance, Yasuda et al. [27] investigated the properties of liquefied sand under undrained monotonic torsional shear conditions up to large strains levels of about 50%. Later, Kiyota et al. [21] conducted undrained cyclic torsional simple shear tests up to double amplitude shear strain exceeding 50%. In the latter tests, a correction for the effects of membrane resistance on measured shear stress was carefully applied [21]. Kiyota et al. [22] reported that the maximum amounts of liquefaction-induced ground displacement observed in relevant model tests and field observations are consistent with the limiting value to initiate strain localization observed in torsional shear tests [21]. Therefore, as long as the shear deformation remains uniform, the results of torsional shear tests can be effectively used to estimate the extent of large deformation that will occur in the field during earthquakes [22].

To evaluate the strength, deformation, and cyclic resistance characteristics of Kpfa pumice under simple shear conditions, monotonic and cyclic undrained shear tests were carried out using a fully automated torsional shear apparatus available at the Institute of Industrial Science, University of Tokyo, Japan (Figure 6a). This device is capable of achieving single-amplitude (i.e., one-way) shear strains γ_{SA} exceeding 50% (or double-amplitude, i.e., peak-to-peak, shear strains γ_{DA} exceeding 100%) on hollow cylindrical

specimens ranging in dimensions from height $H = 200$ mm, inner diameter $d_i = 60$ mm, and outer diameter $d_o = 100$ mm to $H = 300$ mm, $d_i = 90$ mm, and $d_o = 150$ mm [28].

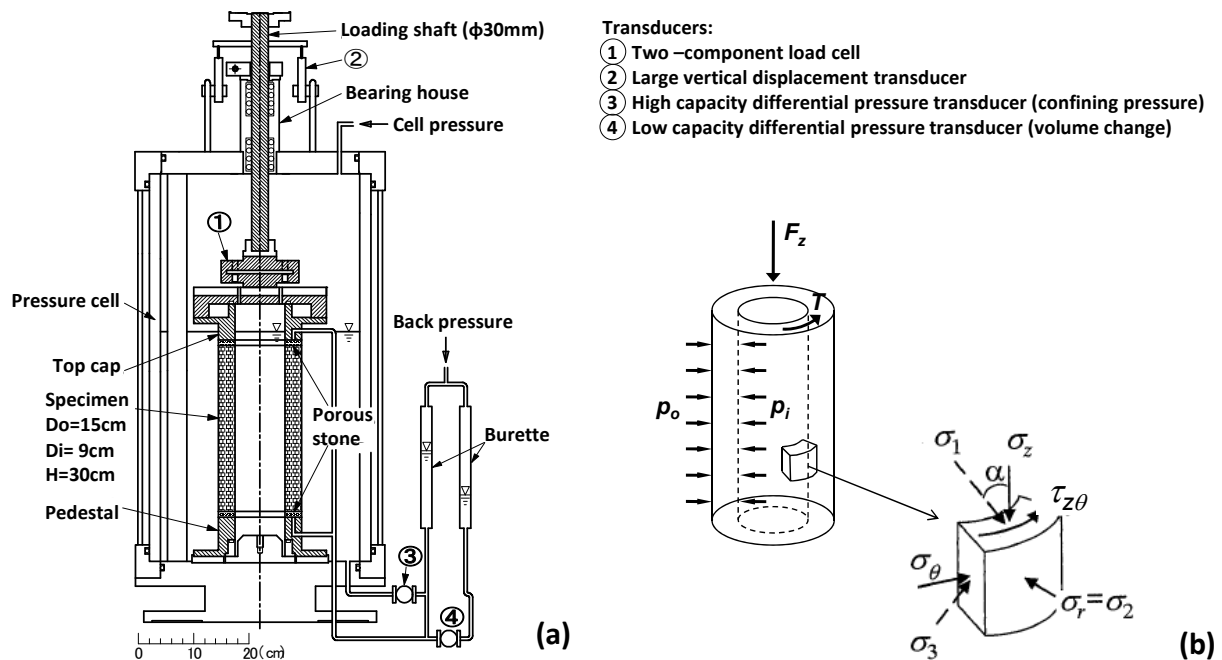


Figure 6. Large-strain torsional shear apparatus used in this study: (a) schematic illustration (adapted from Kiyota et al. [21]; (b) external forces and stress components acting on a hollow cylindrical specimen [29].

In a hollow cylinder torsional shear apparatus, four independent loading components can be applied: vertical load (F_z), torque (T), inner cell pressure (p_i), and outer cell pressure (p_o). The ensuing stress components (Figure 6b), i.e., axial stress (σ_z), radial stress (σ_r), circumferential stress (σ_θ), and torsional shear stress ($\tau_{z\theta}$), can be derived as follows [30]:

$$\sigma_z = \frac{F_z}{\pi(r_o^2 - r_i^2)} + \frac{p_o r_o^2 - p_i r_i^2}{r_o^2 - r_i^2} \quad (1)$$

$$\sigma_r = \frac{p_o r_o + p_i r_i}{r_o + r_i} \quad (2)$$

$$\sigma_\theta = \frac{p_o r_o - p_i r_i}{r_o - r_i} \quad (3)$$

$$\tau = \tau_{z\theta} = \frac{3T}{2\pi(r_o^3 - r_i^3)} \quad (4)$$

where r_o and r_i are the outer and inner radii of the specimen, respectively. The average torsional shear strain ($\gamma_{z\theta}$) is defined as

$$\gamma = \gamma_{z\theta} = \frac{2\theta(r_o^3 - r_i^3)}{2H(r_o^2 - r_i^2)} \quad (5)$$

where θ is the circumferential angular displacement and H is the specimen height.

The average principal stresses σ_1 (major), σ_2 (intermediate), σ_3 (minor), and the mean stress p are given by

$$\left\{ \begin{matrix} \sigma_1 \\ \sigma_3 \end{matrix} \right\} = \frac{\sigma_z + \sigma_\theta}{2} \pm \sqrt{\left(\frac{\sigma_z - \sigma_\theta}{2} \right)^2 + \tau_{z\theta}^2} \quad (6)$$

$$\sigma_2 = \sigma_r \quad (7)$$

$$p = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \quad (8)$$

The device is capable of performing both drained tests, with measurements of volumetric strains, and undrained tests, when drainage is prevented and pore-water pressures u are recorded. Thus, the effective mean stress (p') can be evaluated as

$$p' = p - u \quad (9)$$

It should be noted that, in this study, p_i and p_o were kept equal to each other. Measured shear stress and effective mean stress were corrected in real time for the effects of membrane forces, as reported in Chiaro [29].

3.3. Test Procedure

All the undrained torsional shear tests presented herein were conducted on reconstituted Kpfa pumice specimens. Previous studies have shown that common specimen preparation methods such as wet tamping, wet pluviation, and dry pluviation are not suitable to prepare volcanic soil specimens (e.g., [31]). Wet tamping usually results in particle crushing. During water pluviation, soil grains tend to float instead of sedimenting into the mold, due to air confined in the inner grain pores. Limitations of the air pluviation method have also been encountered because soil particles tend to segregate due to significant variations in individual grain densities. For these reasons, in this study, specimens were prepared in 15 layers of equal thickness using an ad hoc technique, namely, under-compaction dry tamping. This specimen preparation technique is similar to the under-compaction wet tamping technique proposed by Ladd [32] to prepare homogeneous reconstitute sand specimens, with the exception that the material is compacted under dry conditions. Using this technique, (i) the dry mass required to build each layer was weighed out, (ii) loose soil layers were prepared by gently pouring the soil into a split mold, and (iii) the target dry density (ρ_d) of $600 \pm 30 \text{ kg/m}^3$ was achieved by gently tamping the loose soil. The lower layers were compacted to a density slightly less than the desired one as compaction of the upper layers would cause the lower layers to be further compacted and reach the desired density [32]. This allowed preparing homogeneous medium-size hollow cylindrical specimens ($r_o = 75 \text{ mm}$; $r_i = 45 \text{ mm}$; $H = 300 \text{ mm}$) with uniform density while minimizing particle crushing.

Dual-porosity volcanic soils have high internal grain porosity; therefore, their saturation is challenging and requires a thorough de-airing procedure [31,33]. In this study, the double vacuum method proposed by Ampadu [34] was adopted, followed by application of a back pressure of 200 kPa. In this way, a satisfactory degree of saturation was achieved in all tests, with Skempton's B -values greater than 0.95. After completing the saturation process, the pumice specimens were isotropically consolidated to a target effective mean stress (p_0') of 100 kPa. Target values of both density and mean effective stress are representative of the in-situ conditions [7].

Because of the limited amount of soil available, only four strain rate-controlled (0.5%/min shear strain rate) tests were carried out. As summarized in Table 3, they consisted of two monotonic undrained torsional shear tests (Aso#1 and Aso#2) and two cyclic undrained torsional shear tests (Aso#3 and Aso#4).

In monotonic test Aso#2, a static shear stress (τ_{static}) of 25 kPa was applied by pre-shearing the specimen under drained monotonic shear conditions, prior to switching to monotonic undrained shearing, to evaluate the effect of sloping ground conditions [35,36]. In the cyclic tests, only level-ground conditions were considered, and no τ_{static} was applied. During cyclic loading, the loading direction was reversed every time the cyclic shear stress (τ_{cyclic}) amplitude reached a target value of 20 kPa (test Aso#3) or 25 kPa (test Aso#4). To simulate as closely as possible the simple shear stress condition that a soil element

undergoes in the field during earthquake shaking [37,38], in all tests, the top cap was mechanically prevented from displacing vertically during the shearing.

Table 3. Undrained cyclic torsional simple shear tests performed in this study on Kpfa pumice.

Test	Loading Conditions	Dry Density ρ_d (kg/m ³)	Cyclic Shear Stress τ_{cyclic} (kPa)	Static Shear Stress τ_{static} (kPa)
Aso#1	Monotonic	570	—	0
Aso#2		600	—	25
Aso#3	Cyclic	630	20	0
Aso#4		580	25	0

Initial effective mean stress (p_0') = 98 ± 2 kPa.

3.4. Test Results

3.4.1. Monotonic Undrained Shear Response

Figure 7 shows the results of monotonic test Aso#1 in terms of (a) the stress–strain relationship (τ – γ), and (b) the effective stress path (τ – p'). The simulated monotonic undrained torsional shear response of Toyoura sand at 25% and 45% relative density (D_r) under an initial isotropic consolidation stress of $p_0' = 100$ kPa is also shown for comparison. The Toyoura sand results were obtained from element-level test simulations performed with the T-sand model calibrated from large-strain torsional simple shear tests on air-pluviated Toyoura sand (Chiaro et al. [39–41]).

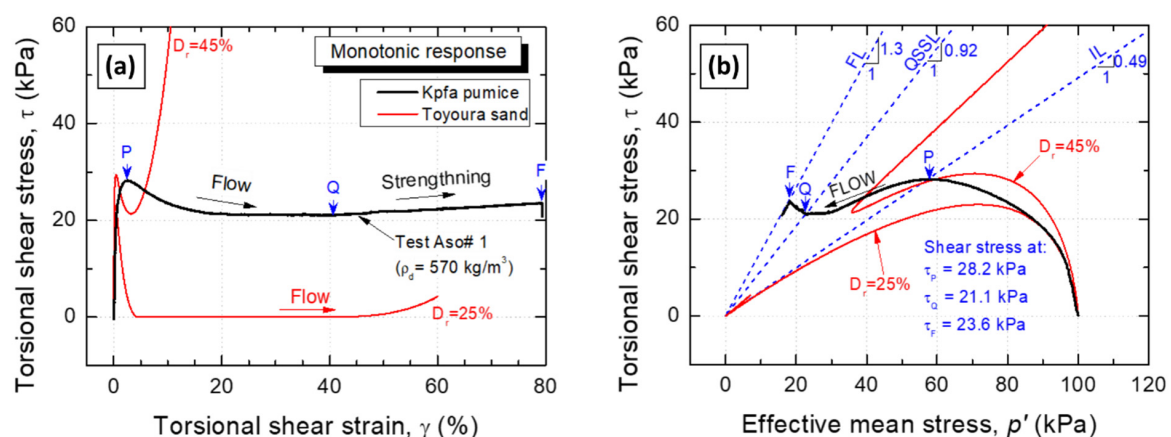


Figure 7. Monotonic undrained torsional shear behavior of Kpfa pumice and its comparison with Toyoura sand: (a) stress–strain relationships; (b) effective stress paths.

From the stress–strain response (Figure 7a) one can note that, after an initial peak stress state (P) at 2.5% shear strain, the specimen exhibits strain-softening behavior (with decreasing shear strength as shear strains increase) to a transient minimum in shear strength (“quasi-steady state” Q [42]) at 40% shear strain, when about 80% of the initial effective confining stress has been lost due to the development of excess pore-water pressures. At larger shear strains, the specimen recovers a fraction of its pre-flow shear strength before reaching the final failure state (F), when a continuous shear band formed all around the specimen. In the effective stress space (Figure 5b), attainment of the peak state (P) is followed by the occurrence of unstable behavior, thus identifying a point on the instability line IL (e.g., [43]). From there, the mean effective stress continues to decrease until reaching the failure envelope (FL).

The simulations on Toyoura sand can be taken as reference for the typical undrained behavior of hard-grained sand (e.g., [44]). Toyoura sand at $D_r = 25\%$ exhibits flow failure, with an initial peak at about 1% shear strain followed by a sudden loss of strength and stiffness. At a nearly zero effective stress state, due to the development of very large excess pore-water pressures, loose Toyoura sand quickly accumulates very large shear strains.

Medium-density ($D_r = 45\%$) Toyoura sand exhibits instead a limited flow type of response. After the mobilization of an initial peak shear strength, a temporary loss in strength is observed (quasi-steady state) with accumulation of about 5% shear strain, followed by strain hardening and rapid recovery of shear strength.

The response observed in test Aso#1 on Kpfa pumice can be likened to that of loose ($D_r = 25\%$) Toyoura sand, i.e., a flow type of response. Nevertheless, the Kpfa pumice specimen developed significant shear strains despite a high value for the effective friction angle mobilized during flow (at state Q) of $\phi_Q' = 42.6^\circ$; this value of ϕ' is significantly larger than those mobilized by loose Toyoura sand during flow (at failure, $\phi_F' = 34^\circ$) and by medium-density Toyoura sand in the quasi-steady state ($\phi_Q' = 31^\circ$), with limited flow [29]. Relatively high effective critical state friction angles ($\phi_{CS}' = 38^\circ$) were also reported by Picarelli et al. [45] in triaxial compression tests on reconstituted (moist-tamped) specimens of sand-sized volcanic air-fall deposits from Italy which, analogously to the test on Kpfa pumice shown in Figure 7, exhibited undrained instability with a flow-type response.

3.4.2. Monotonic Undrained Response under Initial Static Shear

The presence of a driving static shear stress τ_{static} in sloping ground conditions may significantly influence the undrained response of soils, under both monotonic and cyclic shear loading conditions [29]. To evaluate the effects of τ_{static} on the undrained behavior of Kpfa pumice, specimen Aso#2 was pre-sheared by applying an initial shear stress $\tau_{static} = 25$ kPa (Figure 8) before undrained monotonic shearing. In this case, the specimen response was analogous to that observed in test Aso#1 ($\tau_{static} = 0$): after the peak shear strength (P) was mobilized, a marked decrease in shear strength was observed with the development of large strains (P–Q), followed by a small recovery of shear strength before specimen failure (F). Although the shear stresses mobilized at the occurrence of instability (P), at the quasi-steady state (Q), and at failure differ between the two tests, the mobilized effective friction angles at these states were the same (Table 4).

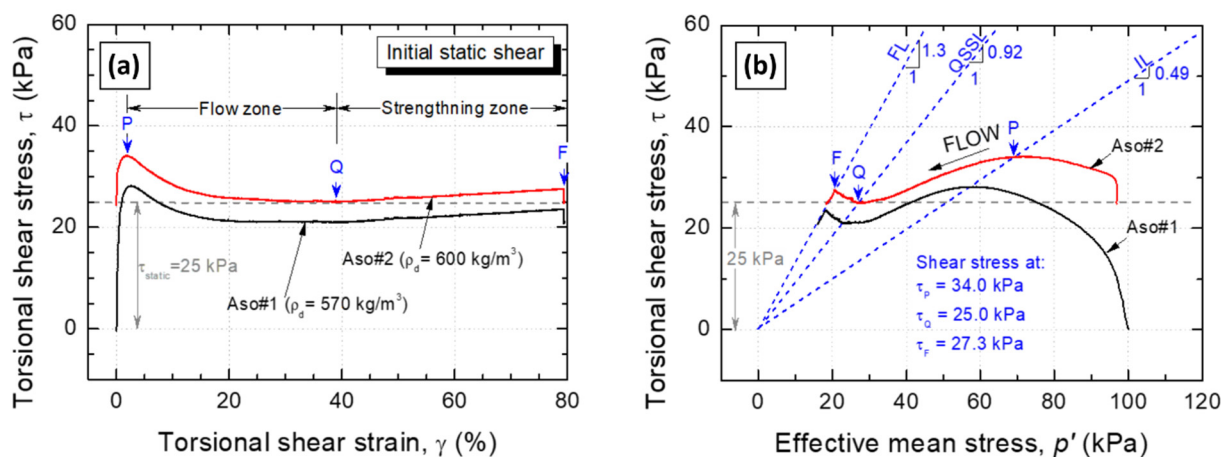


Figure 8. Comparisons between the monotonic undrained behavior of Kpfa pumice in torsional shear tests with and without initial static shear: (a) stress–strain relationships; (b) effective stress paths.

Table 4. Mobilized shear stresses and effective friction angles at relevant states in undrained monotonic torsional simple shear tests with and without initial static shear on Kpfa pumice.

Test	Peak State (P)		Quasi-Steady State (Q)		Failure State (F)	
	τ_P (kPa)	ϕ_P' (°)	τ_Q (kPa)	ϕ_Q' (°)	τ_F (kPa)	ϕ_F' (°)
Aso#1	28.2	26.1	21.1	42.6	23.6	52.4
Aso#2	34.0	26.1	25.0	42.6	27.3	52.4

The presence of a driving shear stress (such as in sloping ground conditions) greater than the quasi-steady state shear strength creates conditions favorable to the generation of landslides with significant runout distances. These could occur after a triggering event, such as an earthquake, causes transient loads greater than the available peak shear strength, after which strain softening would occur, as observed in the two monotonic tests (Aso#1 and Aso#2) presented.

3.4.3. Cyclic Undrained Shear Response

Cyclic undrained torsional shear tests were carried out using two cyclic stress ratio ($CSR = \tau_{cyclic}/p'_0$) amplitudes of 0.20 and 0.25. Figure 9 shows (a) the stress paths (τ – p') with the corresponding (d) stress–strain (τ – γ) response and (c) excess pore-water pressure generation relationships (u_e – γ) for the two tests.

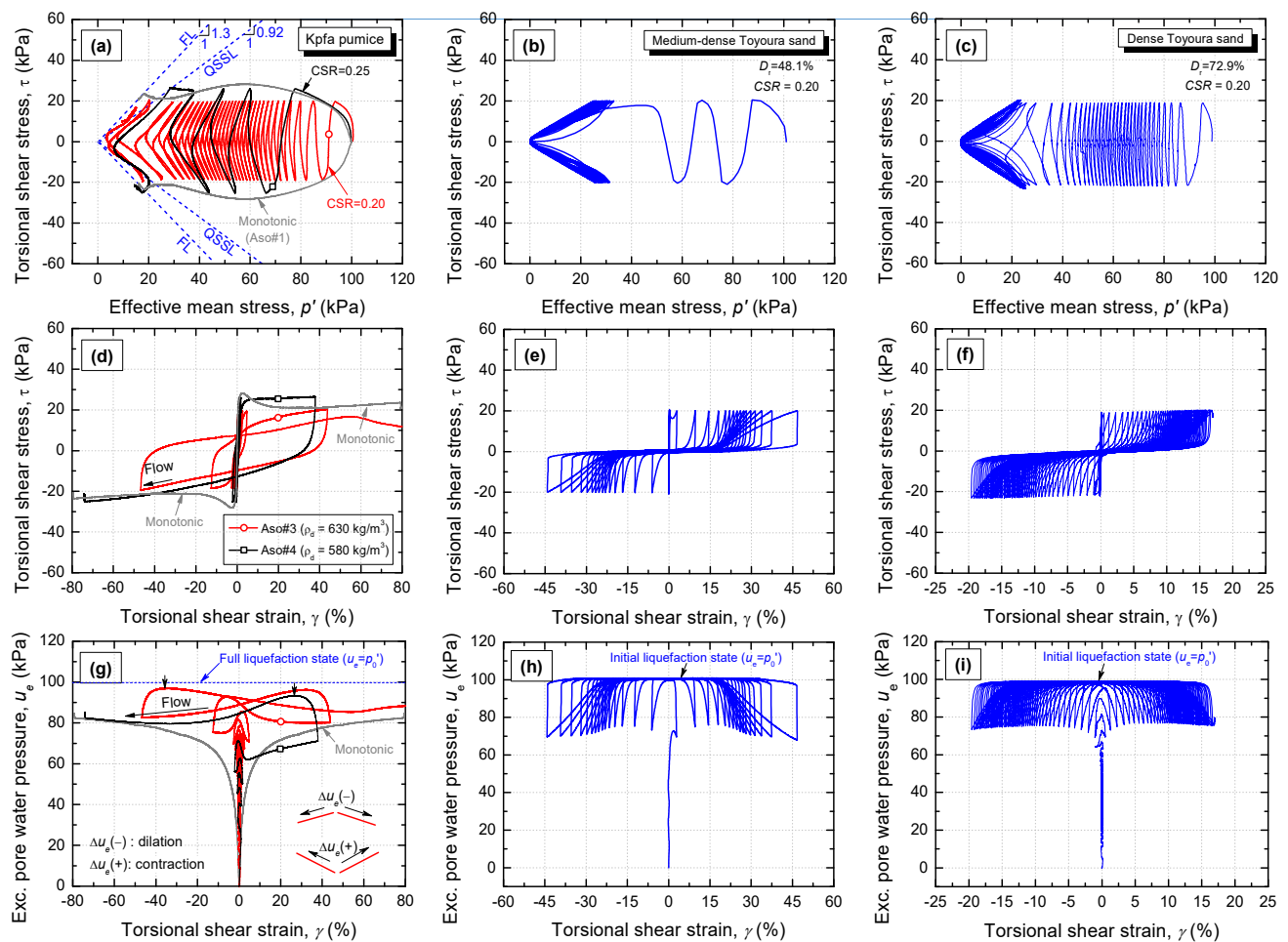


Figure 9. Cyclic undrained torsional shear behavior of Kpfa pumice and air-pluviated Toyoura sand: (a–c) effective stress paths; (d–f) stress–strain relationships; (g–i) excess pore-water pressure generation vs. shear strain.

In both tests, Kpfa pumice exhibited a cyclic liquefaction type of response [37,46]. The effective stress path (Figure 9a) progressively moved toward the left as excess pore-water pressures accumulated in the initial part of the test, when shear strains remained small ($\gamma < 2$ –3%). However, as the excess pore-water pressure ratio $r_u = \Delta u/p'_0$ reached values of 0.75–0.80 (Figure 9g), in both tests, shear strains abruptly increased (“flow”) from $\gamma = 2$ –3% to values larger than $\gamma = 15\%$ in a single half-cycle of loading. Only then, the specimen attained zero- p' conditions (i.e., *initial liquefaction* [47]), with shear strains rapidly growing to $\gamma > 50\%$ in the last stages of the test. Hyodo et al. [31] reported a similar behavior for

loose specimens of Shirasu soil (a crushable volcanic soil from southern Kyushu, Japan) subject to cyclic undrained triaxial loading conditions.

As shown by Figure 9a, in cyclic test Aso#4 with the higher CSR value of 0.25 (greater than the large-strain shear strength τ_F), flow initiated when the effective stress path touched the stress path of the undrained monotonic test with a similar density. In test Aso#3, where the amplitude of the cyclic stress is less than τ_F , the stress path moved toward the failure envelope, and large strains quickly developed when the mobilized friction angle was equal to ϕ_Q' (i.e., the stress path crossed the QSSL).

Figure 9 also presents the results of undrained cyclic torsional shear tests on two air-pluviated specimens of Toyoura sand with relative densities of 48% (Figure 9b–h) and 73% (Figure 9c–i). These sand specimens were tested at the same CSR value of 0.20. In these tests, the Toyoura sand specimens exhibited a cyclic mobility type of response [37,46]. Excess pore-water pressures increased with each loading cycle, causing the effective stress paths (Figure 9b,c) to move leftward until when $p' = 0$ kPa; this process was accompanied by a gradual accumulation of shear strains. This type of response can be contrasted with that exhibited by the Kpfa pumice specimens, which underwent an abrupt and uncontrolled increase in shear strains in the final part of the cyclic tests.

3.4.4. Cyclic Resistance

The resistance to liquefaction or to cyclic strain accumulation of sands is conventionally defined as the number of cycles to attain a reference value of shear strain (γ_{SA} or γ_{DA}) or excess pore water pressure ratio r_u caused by undrained cyclic shear loading. Figure 10 reports the liquefaction resistance curve of Kpfa pumice corresponding to the attainment of $\gamma_{DA} = 7.5\%$. In the same figure, liquefaction resistance curves for medium-density ($D_r = 45\% \pm 5\%$) and dense ($D_r = 67\% \pm 3\%$) air-pluviated Toyoura sand from torsional simple shear tests [13] are reported for comparison. From Figure 10, the cyclic resistance ratio ($CRR_{15} = CSR$ causing liquefaction at 15 cycles of loading) of the pumice was estimated as 0.22. Despite the differences in cyclic behavior described in the previous sections, the CRR_{15} of crushable Kpfa pumice fell between those of hard-grained medium-density to dense Toyoura sand (0.18–0.24).

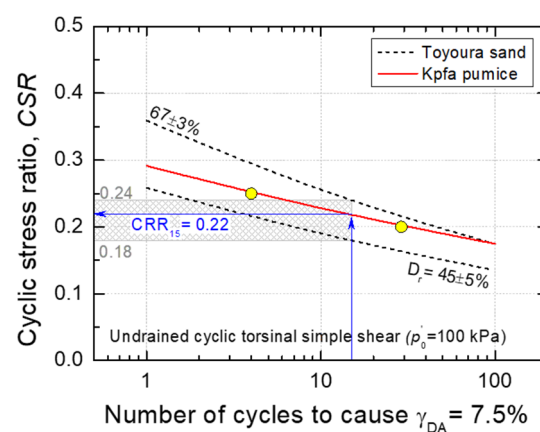


Figure 10. Liquefaction resistance curves for Kpfa pumice (this study) and air-pluviated Toyoura sand [48] attained by undrained cyclic torsional simple shear tests (100 kPa mean effective stress).

4. Liquefaction Assessment of the Takanodai Landslide during the Kumamoto Earthquakes

4.1. Outline of the Procedure

This section presents and discusses liquefaction triggering analyses for the Kpfa pumice layer on the Takanodai hillslope for different seismic events of the 2016 Kumamoto earthquake sequence. The cyclic resistance estimated for the Kpfa pumice in the laboratory with the torsional simple shear tests presented above is compared against estimates of the cyclic stress ratio induced by the earthquake according to a state-of-the-

practice procedure [49,50]. According to the liquefaction assessment procedure of Idriss and Boulanger [50], the factor of safety against liquefaction triggering is given by

$$F_L = 0.9 \frac{CRR_{M_w=7.5, \sigma'_v=100 \text{ kPa}}}{CSR_{M_w, \sigma'_v}} MSF \cdot K_\sigma \cdot K_\alpha \quad (10)$$

where $CRR_{M_w=7.5, \sigma'_v=100 \text{ kPa}}$ is the liquefaction resistance for a given soil layer at 100 kPa initial effective vertical stress for a reference moment magnitude $M_w 7.5$ earthquake (corresponding to approximately 15 cycles of loading by a cyclic sinusoidal shear stress history with constant amplitude), CSR_{M_w, σ'_v} is the seismic demand induced by an earthquake with given moment magnitude M_w in a soil deposit under initial vertical effective stress σ'_v , MSF is a magnitude scaling factor, K_σ and K_α account for the effects of, respectively, initial consolidation stress and static shear stress on liquefaction resistance, and the factor 0.9 accounts for the effects of multi-dimensional shaking (e.g., [51]). At a depth z below the ground surface, CSR can be estimated as

$$CSR_{M_w, \sigma'_v} = \frac{\tau_{\text{cyclic}}}{\sigma'_v} = 0.65 \frac{PGA}{g} \frac{\sigma_v}{\sigma'_v} r_d, \quad (11)$$

where PGA is the geometric mean peak ground acceleration of the two horizontal components of ground motion, g is the acceleration due to gravity (9.81 m/s^2), and σ_v and σ'_v are the initial vertical total and effective stresses, respectively. The nondimensional stress reduction coefficient (r_d) accounts for the flexibility of the soil column and is given by Equation (12) [50].

$$r_d = \exp[\alpha(z) + \beta(z) \cdot M_w] \quad (12)$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \quad (13)$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \quad (14)$$

In this study, MSF is defined by Equation (15) [49].

$$MSF = 6.9 \exp\left(-\frac{M_w}{4}\right) - 0.058 \leq 1.8 \quad (15)$$

For the purposes of liquefaction assessment with Equation (10), the cyclic resistance at 15 cycles of Kpfa pumice obtained from cyclic torsional simple shear ($CRR_{N=15, p'=100 \text{ kPa}}$) was converted from isotropic conditions in the laboratory to anisotropic (i.e., K_0) in-situ stress state conditions [52,53] assuming $K_0 = 0.5$:

$$CRR_{M_w=7.5, \sigma'_v=100 \text{ kPa}} = \frac{(1 + 2K_0)}{3} CRR_{N=15, p'=100 \text{ kPa}} \quad (16)$$

4.2. Evaluation of K_σ for Kpfa Pumice

In quartz sands, an increase in overburden pressure causes a more contractive soil response, resulting in a decrease in CRR . This effect is taken into account in Equation (10) by means of the empirical scaling factor K_σ [54,55] applied to the reference CRR value at $\sigma'_v = 100 \text{ kPa}$. Figure 11 shows K_σ as a function of vertical effective stress for a clean sand with $D_r = 63\%$ (black dashed line), as adopted in the empirical liquefaction triggering procedure of Idriss and Boulanger [50]. For quartz sands, one can note that K_σ gradually decreases from 1.1 to 0.9 as the confining stress increases from 50 kPa to 300 kPa. These values of K_σ can be contrasted against those computed from the cyclic triaxial test data presented by Hyodo et al. [31] for Shirasu crushable volcanic soil at two values of relative density. For the looser Shirasu soil ($D_r = 50\%$), K_σ slightly increases from 0.9 at 50 kPa to 1.1 at 300 kPa; the trends are, therefore, opposite to those for quartz sands. In the case of the denser Shirasu soil ($D_r = 90\%$), CRR_{15} at 50 kPa is about 70% greater than CRR_{15}

at the standard reference pressure of 100 kPa (i.e., $K_\sigma = 1.7$), while, when passing from 100 kPa to 300 kPa of confining stress, CRR_{15} remains unchanged (i.e., $K_\sigma = 1$). The data from Hyodo et al. [31] suggest that values of K_σ based on tests on quartz sand may not be applicable to soils with crushable grains similar to Kpfa pumice.

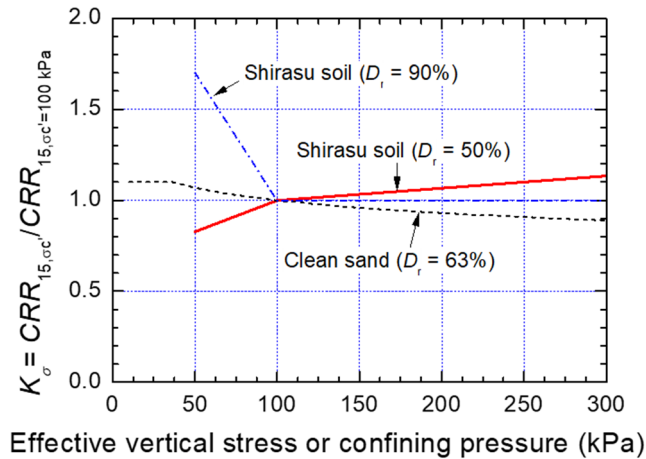


Figure 11. Overburden stress correction factor K_σ for clean sand [50] and for crushable Shirasu volcanic soil (based on experimental data from Hyodo et al. [31]).

In the subsequent liquefaction triggering assessment of the Kpfa pumice layer, to account for the potential effect of particle crushability on CRR of Kpfa pumice at different confining pressure levels, the overburden correction factor reported in Figure 11 for looser Shirasu soil ($D_r = 50\%$) is adopted. For the soil profile shown in Figure 4 and its geometrical variation along the hillslope [2,8], values of effective vertical stress (σ_v') acting on the Kpfa pumice layer at the Takanodai landslide site are estimated to vary in the range of 60 kPa (at the toe of the source area) and 120 kPa (at the hilltop), with corresponding K_σ values between 0.86 and 1.02 (as estimated from Figure 11).

4.3. Evaluation of K_α for Kpfa Pumice

The effect of static shear stress ratio (α) on cyclic resistance ratio (CRR) is accounted for by means of a scaling factor $K_\alpha = CRR_\alpha / CRR_{\alpha=0}$, where CRR_α is the value of CRR for a given value of α , and $CRR_{\alpha=0}$ refers to the reference level ground condition ($\alpha = 0$). Experimental results on different quartz sands at confining pressure less than 300 kPa show that cyclic resistance tends to increase with increasing α in dense sands, and to decrease with increasing α in loose sands. This is shown in Figure 12a using simple shear tests results [23,25] and in Figure 12b using torsional simple shear test results [28]. The authors are not aware of experimental data to estimate values of K_α for crushable volcanic soils; for this reason, the subsequent analyses employ the K_α values for Toyoura sand as simulated by the T-sand model [39–41] for air-pluviated Toyoura sand with $D_r = 62\%$ in torsional simple shear conditions at $\sigma_v' = 100$ kPa. In these conditions, $CRR_{15, \alpha=0} = 0.22$ for both Toyoura sand and Kpfa pumice (Figure 10). The relationship between K_α and α is shown in Figure 13a and is consistent with the laboratory test data reproduced in Figure 12b. One can observe from Figure 13a that K_α first decreases with increasing α (detrimental effect of τ_{static}) and then increases (beneficial effect of τ_{static}). The resulting values of CRR_{15} after application of the scaling factor K_α are plotted in Figure 13b.

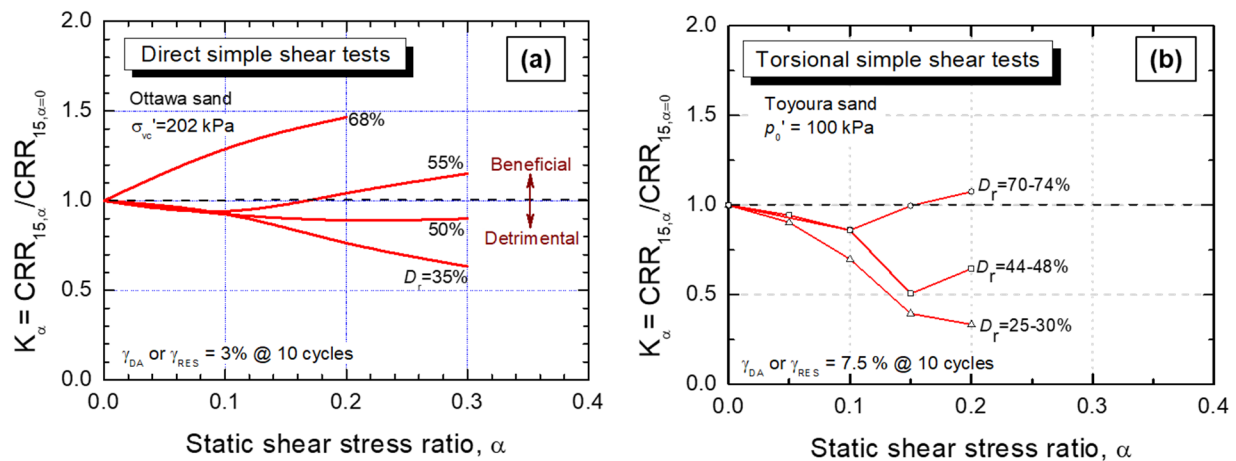


Figure 12. Relationship between K_α factor and static shear stress ratio: (a) simple shear tests [23,25]; (b) torsional simple shear tests [28].

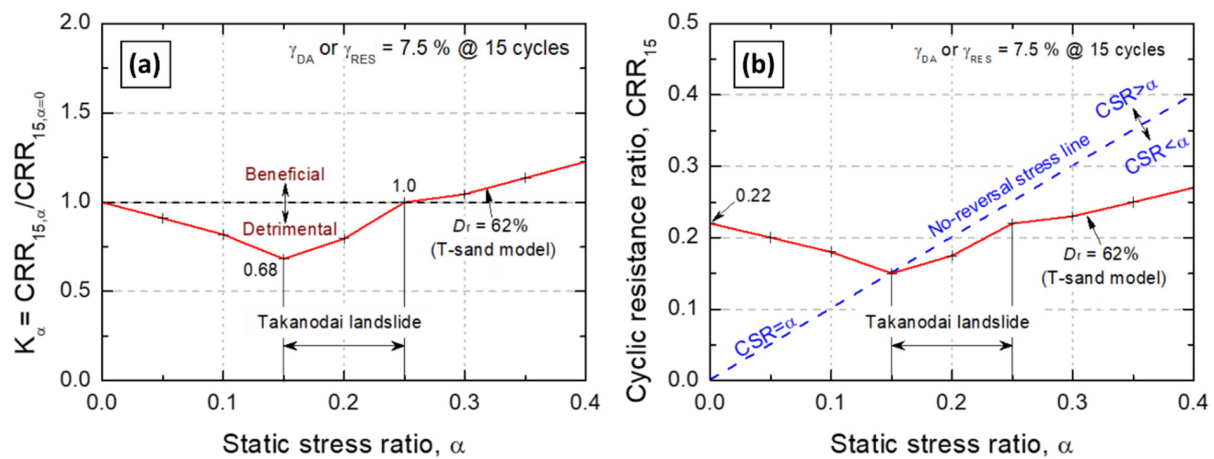


Figure 13. Cyclic resistance of Kpfa pumice evaluated by T-sand model: (a) K_α vs. α relationship; (b) CRR_{15} vs. α relationship.

Assuming infinite slope conditions, α can be calculated as follows [56]:

$$\alpha_{\text{field}} = \frac{\sigma_v}{[(1 + 2K_0)/3]\sigma'_v} \tan \beta, \quad (17)$$

where β is the angle of inclination of the slope. For the Takanodai landslide site, considering $\beta = 6^\circ$ and 10° , Equation (17) returns values of α for the Kpfa pumice layer of 0.15 and 0.25, respectively; the corresponding K_α values vary between 0.68 and 1.00 (Figure 13a).

4.4. Liquefaction Assessment: Results and Discussion

Figures 14 and 15, and Table 5 summarize the results of the liquefaction assessment for the two largest events of the Kumamoto earthquake sequence, namely, the $M_w 6.2$ 14 April 2016 and the $M_w 7.0$ 16 April 2016. To compare CSR and CRR values, these should be referred to the same earthquake magnitude (i.e., number of equivalent loading cycles). On this purpose, the CSR^* values shown in Table 5 and Figures 14 and 15 were obtained by scaling CSR given by Equation (11) as follows:

$$CSR^* = CSR_{M_w=7.5, \sigma'_v=100\text{kPa}} = \frac{CSR_{M_w, \sigma'_v}}{MSF \cdot K_\sigma} \quad (18)$$

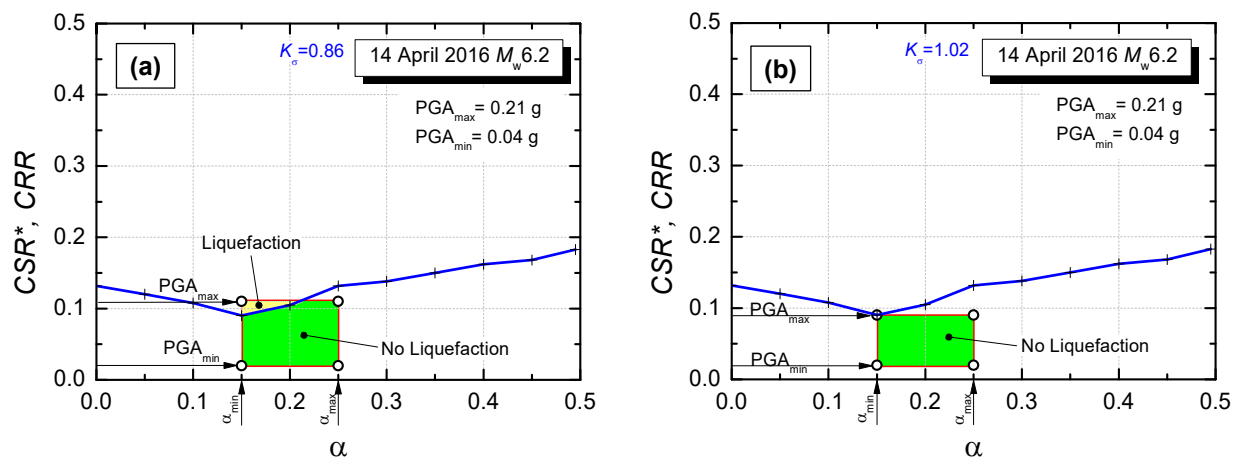


Figure 14. Results of liquefaction triggering analyses for the Takanodai landslide site considering the strong motion records of the 14 April 2016 M_w 6.2 for (a) $K_\sigma = 0.86$ (toe of source area) and (b) $K_\sigma = 1.02$ (hilltop). The shaded areas in each plot indicate the range of estimated values of α vs. CSR^* used in the analyses to account for uncertainty in the input parameters.

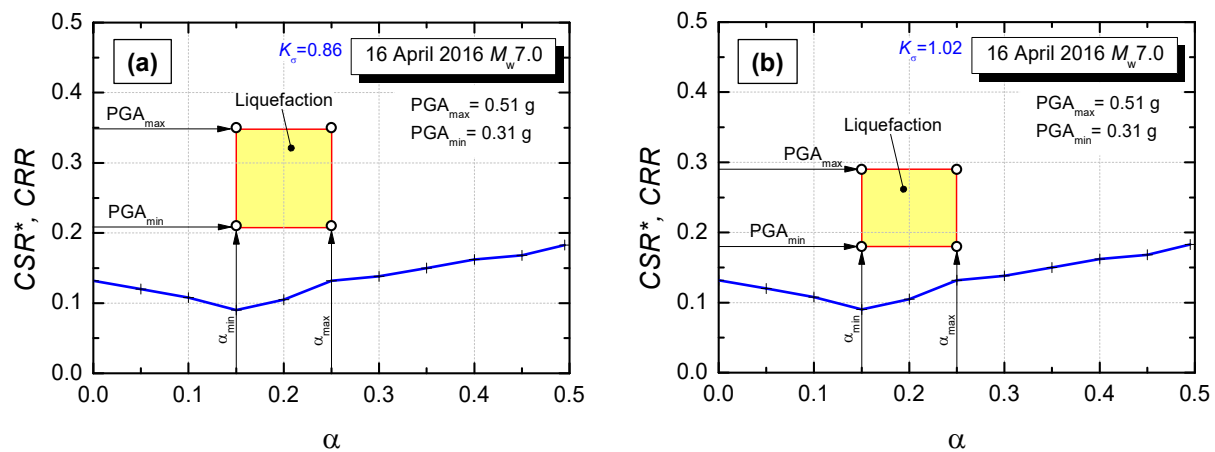


Figure 15. Results of liquefaction triggering analyses for the Takanodai landslide site considering the strong motion records of the 16 April 2016 M_w 7.0 for (a) $K_\sigma = 0.86$ (toe of source area) and (b) $K_\sigma = 1.02$ (hilltop). The shaded areas in each plot indicate the range of estimated values of α vs. CSR^* used in the analyses to account for uncertainty in the input parameters.

Table 5. Summary of liquefaction triggering analyses.

Earthquake	Station	PGA (g)	$CRR^\dagger \cdot K_\alpha$	CSR^*	F_L	CSR^*	F_L
			$\alpha = 0.15\text{--}0.25$	$K_\sigma = 0.86$		$K_\sigma = 1.02$	
14 April 2016 M_w 6.2 ($MSF = 1.41$)	KMM004	0.04	0.09–0.13	0.02	4.5–6.5	0.02	4.5–6.5
	KMM005	0.21	0.09–0.13	0.11	0.8–1.2	0.09	1.0–1.4
16 April 2016 M_w 7.0 ($MSF = 1.14$)	KMM004	0.31	0.09–0.13	0.21	0.4–0.6	0.18	0.5–0.7
	KMM005	0.51	0.09–0.13	0.35	0.3–0.4	0.29	0.3–0.4

[†] Includes the effects of multidirectional shaking and anisotropic in-situ stress; * CSR from Equation (18).

For both seismic events, CSR^* values were computed considering both lower-bound (Figures 14a and 15a) and upper-bound values (Figures 14b and 15b) of K_σ . CRR values were scaled to include the effects of multidirectional shaking and of anisotropic in situ stress conditions (Equations (10) and (16)).

For both earthquake scenarios, in the calculation of CSR , the geometric mean PGA values recorded at the stations KMM005 (located outside the Mt. Aso Caldera, above the rupture surface) and KMM004 (located within the Mt. Aso Caldera, along the faultline

projection of the main event) (Figure 1) were selected to account for uncertainty in the estimate of PGA values in the absence of direct strong motion measurements at the site. In this case of the 14 April 2016 foreshock, the source-to-site distance of the landslide site is comprised between the source-to-site distances of the two stations. For the main shock event, due to its location in the proximity of the northern end of the rupture surface, the site likely underwent fault-rupture directivity effects (which were noted in strong motion records, e.g., [16–18]) with ground accelerations greater than those recorded at station KMM004 (which is located further away from the rupture surface).

Figure 14 shows that, for upper-bound estimates of K_σ (Figure 15b), the cyclic demand associated with the $M_w 6.2$ 14 April 2016 earthquake, represented by the green shading, did not exceed the cyclic resistance of Kpfa pumice, represented by the blue line, and it was, therefore, insufficient to trigger liquefaction. The factor of safety against liquefaction (F_L) varies between 1.0 and 6.5 (Table 5), indicating that the seismic demand was significantly lower than the cyclic resistance of the Kpfa pumice deposit. Yet, for the lower-bound estimates of K_σ (Figure 15a), the analyses indicate that liquefaction was still possible under strong accelerations ($F_L = 0.8–6.5$), but only if the actual CSR at the landslide site exceeded 0.09. The $M_w 7.0$ 16 April 2016 earthquake, however, resulted in greater $PGAs$ which were sufficient to trigger liquefaction, as shown in Figure 15. Concurring low values of K_σ and K_α (Figure 15a) resulted in CSR values significantly higher than CRR of the pumice layer, as indicated by the yellow shading ($F_L = 0.3–0.7$). It is, therefore, likely that the combination of large inertial forces induced by the main shock earthquake and the presence of a driving initial static shear stress triggered liquefaction in the Kpfa pumice layer, which subsequently initiated a flow-type landslide. The results of these analyses appear, therefore, be consistent with the observed evidence, with the slope at Takanodai remaining stable during the first event, but failing with catastrophic consequences during the main shock.

One should note some limitations of the present analyses. The estimate of CRR for the pumice layer employed in the calculations was derived from laboratory tests on reconstituted soil specimens, prepared in the laboratory. However, soil fabric and structure (i.e., the arrangement of soil particles at the micro and macroscale, e.g., [57]) are primary factors influencing the cyclic strength of soils [58], and they are uniquely associated with the adopted specimen preparation method or the in-situ formation process of a soil deposit. The use of high-quality sampling technique which could provide soil specimens suitable for testing in the laboratory (e.g., [59,60]) would, therefore, provide a significant contribution to the present analyses.

The results of the analyses are significantly influenced by the values adopted for the factors K_σ and K_α for Kpfa pumice, which, in this case, were based on studies on other soils: a different crushable volcanic soil for K_σ , and a hard-grained sand for K_α . No studies have addressed so far (at the best of the authors' knowledge) the influence of static shear stress on the cyclic response of pumiceous soils. Such investigations would provide a useful contribution for the assessment of the seismic stability of slopes of these peculiar soil deposits.

5. Conclusions

This study re-examined the case study of the Takanodai landslide that occurred in Minami Aso during the 2016 Kumamoto earthquakes. It employed the results of advanced laboratory tests on a pumiceous soil (Kpfa pumice) which previous investigations had identified as the critical layer associated with the slope failure, and it performed liquefaction triggering analyses to assess the in situ performance of the critical layer for two earthquake scenarios of the 2016 seismic sequence. The following conclusions can be drawn:

- **Laboratory investigations**

Large-strain undrained torsional shear tests confirmed that Kpfa pumice has the tendency to show post-peak flow-type failure behavior characterized by an abrupt development of large shear strains exceeding 50% or more when subjected to monotonic shearing.

The presence of an initial driving shear stress may significantly contribute to the observed flow-type failure and associated large deformation.

Under cyclic shear stress condition, tested specimens exhibited a progressive build-up of excess pore water pressure and shear strains. Following the attainment of shear strains of 2–3%, the specimens exhibited a flow-type failure with a rapid development of large (>15%) shear strains.

- **Liquefaction triggering analyses**

The results of liquefaction triggering analyses are consistent with the response observed in situ, when the slope at Takanodai remained stable during the M_w 6.2 14 April 2016 earthquake but failed as a result of the M_w 7.0 16 April 2016 earthquake. The concurring effect of high cyclic stress ratios ($CSR = 0.21$ – 0.35) induced by the earthquake and static shear stress ratios ($\alpha = 0.15$ – 0.25) were the critical factors—leading to the seismic demand exceeding the liquefaction resistance of the Kpfa pumice layer ($CRR = 0.09$ – 0.13)—responsible for the observed liquefaction-induced flow-type landslide.

- **Future research needs**

The experimental results and triggering analyses highlight the role that factors such as K_σ and K_α play in the undrained cyclic response and seismic slope stability of pumiceous deposits. Nevertheless, very few experimental studies have been performed on these topics, and extensive research work is, therefore, needed in these largely unexplored research areas.

Author Contributions: Conceptualization, G.C. and T.K.; methodology, G.C., T.K., M.U. and C.C.; laboratory investigation, M.U.; writing—original draft preparation, G.C. and C.C.; writing—review and editing, T.K. and M.U.; project administration, G.C. and T.K.; funding acquisition, G.C. and T.K. All authors read and agreed to the published version of the manuscript.

Funding: This research was funded by the NZSEE through the “Learning from Earthquake” Kumamoto-mission, and the Japan Science and Technology Agency (JST) through the J-Rapid Program (International Urgent Collaborative Projects Regarding the 2016 Kumamoto Earthquake).

Data Availability Statement: Not applicable.

Conflicts of Interest: The authors declare no conflict of interest. The funders had no role in the design of the study; in the collection, analyses, or interpretation of data; in the writing of the manuscript; or in the decision to publish the results.

References

1. Mukunoki, T.; Kasama, T.; Murakami, S.; Ikemi, H.; Ishikura, R.; Fujikawa, T.; Yasufuka, N.; Kitazono, Y. Reconnaissance report on geotechnical damage caused by an earthquake with JMA seismic intensity 7 twice in 28h, Kumamoto, Japan. *Soils Found.* **2016**, *56*, 947–964. [\[CrossRef\]](#)
2. Dang, K.; Sassa, K.; Fukuoka, H.; Sakai, N.; Sato, Y.; Takara, K.; Quang, L.H.; Loi, D.H.; Tien, P.V.; Ha, N.D. Mechanism of two rapid and long runout landslides in the 16 April 2016 Kumamoto earthquake using a ring-shear apparatus and computer simulation (LS-RAPID). *Landslides* **2016**, *13*, 1525–1534. [\[CrossRef\]](#)
3. Kiyota, T.; Ikeda, T.; Konagai, K.; Shiga, M. Geotechnical damage caused by the 2016 Kumamoto Earthquake, Japan. *Int. J. Geoeng. Case Hist.* **2017**, *4*, 78–95.
4. Chiaro, G.; Alexander, G.; Brabhakaran, P.; Massey, C.; Koseki, J.; Yamada, S.; Aoyagi, Y. Reconnaissance report on geotechnical and geological aspects of the 2016 Kumamoto Earthquake, Japan. *Bull. N. Z. Soc. Earthq. Eng.* **2017**, *50*, 365–393.
5. Chigira, M.; Suzuki, T. Prediction of earthquake-induced landslides of pyroclastic fall deposits. In *Landslides and Engineered Slopes. Experience, Theory and Practice*; CRC Press: Boca Raton, FL, USA, 2016; pp. 93–100.
6. Wang, G. Fluidized landsliding phenomena during earthquakes. In Proceedings of the JTC1 Workshop on Advances in Landslide Understanding, Barcelona, Spain, 24–26 May 2017; p. 4.
7. Chiaro, G.; Umar, M.; Kiyota, T.; Massey, C. The Takanodai landslide, Kumamoto, Japan: Insights from post-earthquake field observations, laboratory tests and numerical analyses. *ASCE Geotech. Spec. Publ.* **2018**, *293*, 98–111.
8. Chiaro, G.; Chew, K.; Kim, J.S. Numerical analyses of the earthquake-induced Takanodai landslide, Kumamoto, Japan. In Proceedings of the Conference on GIS and Geoinformation Zoning for Disaster Mitigation, Auckland, New Zealand, 23–24 November 2018; pp. 47–54.

9. Chiaro, G.; Kiyota, T.; Umar, M. Undrained monotonic and cyclic torsional simple shear behavior of the Aso pumiceous soil deposits. In Proceedings of the 7th International Conference on Earthquake Geotechnical Engineering, Rome, Italy, 17–20 June 2019; pp. 1754–1761.
10. Umar, M.; Chiaro, G.; Takashi, K.; Miyamoto, H. Monotonic and cyclic undrained behavior of Kumamoto-Aso pumice soil by triaxial and torsional shear tests. In Proceedings of the 16th European Conference on Earthquake Engineering, Thessaloniki, Greece, 18–21 June 2018; Paper No. 11577. p. 11.
11. Hazarika, H.; Sumartini, W.O.; Kokusho, T.; Kochi, Y.; Ishibashi, S.; Yamamoto, S.; Matsumoto, D. Investigations on mechanism of landslides during the 2016 Kumamoto Earthquake, Japan. In Proceedings of the 7th International Conference on Earthquake Geotechnical Engineering, Rome, Italy, 17–20 June 2019; pp. 821–832.
12. Kasama, K.; Furukawa, Z.; Yasufuku, N. Cyclic shear property and seismic runout analysis for pumice fall deposit. *Soil Dyn. Earthq. Eng.* **2021**, *143*, 106588. [\[CrossRef\]](#)
13. Goto, S.; Okada, K. Monotonic and cyclic behavior of tephra layer landslide at Takanodai from the 2016 Kumamoto earthquake. In *Understanding and Reducing Landslide Disaster Risk*; Tiwari, B., Sassa, K., Bobrowsky, P.T., Takara, K., Eds.; Springer: Cham, Switzerland, 2020; pp. 357–364.
14. Geological Survey of Japan, AIST Website. Available online: <https://gbank.gsj.jp/geonavi/geonavi.php#12,32.87856,130.99942> (accessed on 28 July 2022).
15. Geological Survey of Japan. Available online: <https://www.gsj.jp/hazards/earthquake/kumamoto2016/geonavi-link.html> (accessed on 28 July 2022).
16. Asano, K.; Iwata, T. Revisiting the source process of the mainshock of the 2016 Kumamoto earthquake and implications for the generation of near-fault ground motions and forward-directivity pulse. *Bull. Seism. Soc. Am.* **2021**, *111*, 2426–2440. [\[CrossRef\]](#)
17. Kubo, H.; Suzuki, W.; Aoi, S.; Sekiguchi, H. Source rupture process of the 2016 Kumamoto, Japan, earthquakes estimated from strong-motion waveforms. *Earth Planets Space* **2016**, *68*, 161. [\[CrossRef\]](#)
18. Yoshida, K.; Miyakoshi, K.; Somei, K.; Irikura, K. Source process of the 2016 Kumamoto earthquake (Mj7.3) inferred from kinematic inversion of strong-motion records. *Earth Planets Space* **2017**, *69*, 64. [\[CrossRef\]](#)
19. K-net. Available online: <https://www.kyoshin.bosai.go.jp/> (accessed on 28 July 2022).
20. Paudel, P.; Omura, H.; Kubota, T.; Inoue, T. Spatio-temporal patterns of historical shallow landslides in a volcanic area, Mt. Aso, Japan. *Geomorphology* **2007**, *88*, 21–33. [\[CrossRef\]](#)
21. Kiyota, T.; Sato, T.; Koseki, J.; Mohammad, A. Behavior of liquefied sands under extremely large strain levels in cyclic torsional shear tests. *Soils Found.* **2008**, *48*, 727–739. [\[CrossRef\]](#)
22. Kiyota, T.; Koseki, J.; Sato, T. Comparison of liquefaction-induced ground deformation between results from undrained cyclic torsional shear tests and observations from previous model tests and case studies. *Soils Found.* **2010**, *50*, 421–429. [\[CrossRef\]](#)
23. Vaid, Y.P.; Finn, W.D.L. Static shear and liquefaction potential. *J. Geotech. Eng. Div. ASCE* **1979**, *105*, 1233–1246. [\[CrossRef\]](#)
24. Tatsuoka, F.; Muramatsu, M.; Sasaki, T. Cyclic undrained stress-strain behavior of dense sands by torsional simple shear stress. *Soils Found.* **1982**, *22*, 55–70. [\[CrossRef\]](#)
25. Boulanger, R.W.; Seed, R.B.; Chan, C.K.; Seed, H.B.; Sousa, J.B. *Liquefaction Behavior of Saturated and under Uni-Directional and Bi-directional Monotonic and Cyclic Simple Shear Loading*; University of California Berkley: Berkeley, CA, USA, 1991.
26. Cappellaro, C.; Cubrinovski, M.; Bray, D.J.; Chiaro, G.; Riemer, M.; Stringer, M. Liquefaction resistance of Christchurch sandy soils from direct simple shear tests. *Soil Dyn. Earthq. Eng.* **2021**, *141*, 106489. [\[CrossRef\]](#)
27. Yasuda, S.; Nagase, N.; Kiku, H.; Uchida, Y. The mechanisms and a simplified procedure for analyses of permanent ground displacement due to liquefaction. *Soils Found.* **1992**, *32*, 149–160. [\[CrossRef\]](#)
28. Umar, M.; Chiaro, G.; Kiyota, T.; Ullah, N. Deformation and cyclic resistance of sand in large-strain undrained torsional shear tests with initial static shear stress. *Soils Found.* **2021**, *61*, 765–781. [\[CrossRef\]](#)
29. Chiaro, G. Cyclic resistance and large deformation characteristics of sands under sloping ground conditions: Insights from large-strain torsional simple shear tests. In *Latest Developments in Geotechnical Earthquake Engineering and Soil Dynamics, Springer Transactions in Civil and Environmental Engineering*; Sitharam, T.G., Ed.; Springer: Singapore, 2021; pp. 101–131.
30. Hight, D.W.; Gens, A.; Symes, M.J. The development of a new hollow cylinder apparatus for investigating the effects of principal stress rotation in soils. *Geotechnique* **1983**, *33*, 355–383. [\[CrossRef\]](#)
31. Hyodo, M.; Hyde, A.F.L.; Aramaki, N. Liquefaction of crushable soils. *Geotechnique* **1998**, *48*, 527–543. [\[CrossRef\]](#)
32. Ladd, R. Preparing tests specimens using undercompaction. *Geotech. Test. J.* **1978**, *1*, 16–23.
33. Pender, M.J.; Wesley, L.D.; Larkin, T.J.; Pranjoto, S. Geotechnical properties of a pumice sand. *Soils Found.* **2006**, *46*, 69–81. [\[CrossRef\]](#)
34. Ampadu, S.I.K. Undrained Behavior of Kaolin in Torsional Simple Shear. PhD Thesis, University of Tokyo, Tokyo, Japan, 1991.
35. Chiaro, G.; Koseki, J.; Sato, T. Effects of initial static shear on liquefaction and large deformation properties of loose saturated Toyoura sand in undrained cyclic torsional shear. *Soils Found.* **2012**, *52*, 498–510. [\[CrossRef\]](#)
36. Chiaro, G.; Kiyota, T.; Koseki, J. Strain localization characteristics of loose saturated Toyoura sand in undrained cyclic torsional shear tests with initial static shear. *Soils Found.* **2013**, *53*, 23–34. [\[CrossRef\]](#)
37. Seed, H.B.; Lee, K.L. Liquefaction of saturated sands during cyclic loading. *J. Soil Mech. Found. Div.* **1966**, *92*, 105–134. [\[CrossRef\]](#)
38. Finn, W.D.L. Aspects of constant volume cyclic simple shear. In *Advances in the Art of Testing Soils Under Cyclic Conditions*; Khosla, V., Ed.; ASCE: Reston, VI, USA, 1985; pp. 74–98.

39. Chiaro, G.; Koseki, J.; De Silva, L.I.N. A density- and stress-dependent elasto-plastic model for sands subjected to monotonic torsional shear loading. *Geotech. Eng. J. SEAGS* **2013**, *44*, 18–26.
40. Chiaro, G.; De Silva, L.I.N.; Koseki, J. Modeling the effects of static shear on the undrained cyclic torsional simple shear behavior of liquefiable sand. *Geotech. Eng. J. SEAGS* **2017**, *48*, 1–9.
41. Chiaro, G.; De Silva, N.L.I.; Koseki, J. Modeling of liquefaction and large-strain response of sand in sloping ground. In *Modeling in Geotechnical Engineering*; Samui, P., Kumari, S., Makarov, V., Kurup, P., Eds.; Elsevier: Amsterdam, The Netherlands, 2021; pp. 365–391.
42. Ishihara, K.; Tatsuoka, F.; Yasuda, S. Undrained deformation and liquefaction of sand under cyclic stresses. *Soils Found.* **1975**, *15*, 29–44. [[CrossRef](#)]
43. Sladen, J.A.; D'Hollander, R.D.; Krahn, J. The liquefaction of sands, a collapse surface approach. *Can. Geotech. J.* **1985**, *22*, 564–578. [[CrossRef](#)]
44. Yoshimine, M.; Ishihara, K. Flow potential of sand during liquefaction. *Soils Found.* **1998**, *38*, 189–198. [[CrossRef](#)]
45. Picarelli, L.; Olivares, L.; Lampitiello, S.; Darban, R.; Damiano, E. The undrained behaviour of an air-fall volcanic ash. *Geosciences* **2020**, *10*, 60. [[CrossRef](#)]
46. Seed, H.B. Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes. *J. Geotech. Eng. Div.* **1979**, *105*, 201–255. [[CrossRef](#)]
47. Ishihara, K. Liquefaction and flow failure during earthquakes. *Géotechnique* **1993**, *4*, 351–415. [[CrossRef](#)]
48. Chiaro, G.; Kiyota, T.; Miyamoto, H. Liquefaction potential and large deformation properties of Christchurch liquefied sand subjected to undrained cyclic torsional simple shear loading. In Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering, Seoul, Korea, 17–22 September 2017; pp. 1497–1500.
49. Seed, H.B.; Idriss, I.M. Simplified procedure for evaluating soil liquefaction potential. *J. Soil Mech. Found. Div. ASCE* **1971**, *97*, 1249–1273. [[CrossRef](#)]
50. Idriss, I.M.; Boulanger, R.W. Soil Liquefaction during earthquakes. In *Engineering Monographs on Earthquake Criteria, Structural Design, and Strong motion Records*; Earthquake Engineering Research Institute: Oakland, CA, USA, 2008.
51. Pyke, R.M.; Chan, C.K.; Seed, H.B. *Settlement and Liquefaction of Sands under Multi-Directional Shaking*; Report No. EERC 74-2; Earthquake Engineering Research Center, University of California at Berkeley: Berkeley, CA, USA, 1974.
52. Ishihara, K.; Takatsu, H. Effects of overconsolidation and K_0 , conditions on the liquefaction characteristics of sands. *Soils Found.* **1979**, *19*, 59–68. [[CrossRef](#)]
53. Ishihara, K.; Yamazaki, A.; Haga, K. Liquefaction of K_0 consolidated sand under cyclic rotation of principal stress direction with lateral constraint. *Soils Found.* **1985**, *5*, 63–74. [[CrossRef](#)]
54. Harder, L.F.; Boulanger, R.W. Application of K_σ and K_α correction factors. In Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, UT, USA, 5–6 January 1996; Youd, T.L., Idriss, I.M., Eds.; Technical Report NCEER-97-0022. National Center for Earthquake Engineering Research, SUNY: Buffalo, NY, USA, 1997; pp. 167–190.
55. Vaid, Y.P.; Sivathayalan, S. Static and cyclic liquefaction potential of Fraser delta sand in simple shear and triaxial tests. *Can. Geotech. J.* **1996**, *33*, 281–289. [[CrossRef](#)]
56. Chiaro, G. Deformation Properties of Sand with Initial Static Shear in Undrained Cyclic Torsional Shear Tests and Their Modeling. Ph.D. Thesis, Department of Civil Engineering, University of Tokyo, Tokyo, Japan, 2010; p. 310.
57. Verdugo, R.; Castillo, P.; Briceño, L. Initial soil structure and steady state strength. In Proceedings of the 1st International Conference on Earthquake Geotechnical Engineering, Tokyo, Japan, 14–16 November 1995.
58. Mulilis, J.P.; Seed, H.B.; Chan, C.K.; Mitchell, J.K.; Arulanandan, K. Effects of sample preparation on sand liquefaction. *J. Geotech. Eng. Div.* **1977**, *103*, 91–108. [[CrossRef](#)]
59. Umehara, Y.; Chiaro, G.; Kiyota, T.; Hosono, Y.; Yaguira, Y.; Chiba, H. Effectiveness of ‘Gel-Push’ sampling technique to retrieve undisturbed sandy specimens for liquefaction test. In Proceedings of the 6th International Conference on Earthquake Geotechnical Engineering, Christchurch, New Zealand, 1–4 November 2015; p. 8.
60. Mori, K.; Sakai, K. The GP sampler: A new innovation in core sampling. In Proceedings of the 5th International Conference on Geotechnical and Geophysical Site Characterization, Gold Coast, Australia, 5–9 September 2016; pp. 99–124.