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Some fluidized landslides triggered by the 2011 Tohoku Earthquake (M9.0), Japan

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1 Abstract

2 The 2011 Tohoku Earthquake off the Pacific coast of Japan generated a large tsunami and many landslides,
3 resulting in a great number of casualties. Although almost all casualties resulted from the tsunami, some
4 long-travel, fluidized small-scale landslides also killed 13 people. After the earthquake, we surveyed seven of
5 these catastrophic landslides triggered by the earthquake. We found that most of them have nearly identical
6 geological features, with slopes consisting of pyroclastic deposits formed at different times, and with a palaeosol
7 layer that outcropped in most cases after the landslide. Above the palaeosol there are layers of pumice and
8 scoria. The palaeosol had a natural moisture content of ~160%, and the pumice and scoria a moisture content of
9 ~145%. From field observations we concluded that the sliding surface originated in the very upper part of
10 palaeosol, and liquefaction occurred in both layers, resulting in the fluidization of displaced landslides. To
11 examine the trigger and movement mechanism of these landslides, we monitored the ground motion of one
12 landslide area during the many aftershocks, and compared the results with records obtained by a national seismic
13 station nearby. We inferred that strong seismic motion occurred in the landslide area during the main shock. We
14 sampled the palaeosol and pyroclastic deposits, and performed undrained static/cyclic shear tests on the materials
15 both in a saturated state and at natural moisture content. The results indicate that high pore-water pressure
16 generated, resulting in decreased shear strength even in samples with the natural moisture content. The shear
17 strength of the palaeosol lowered to a very small value with continuous increase of shear rate, enabling the high
18 mobility of the displaced landslide materials.

19

20 Keywords: Earthquake-induced landslide; the 2011 Tohoku earthquake; Pore-water pressure; Partially saturated
21 soils; Strong ground motion

22

23 1. Introduction

24 An earthquake of magnitude 9.0 (M_w) occurred at 14:46 JST on 11 March 2011, off the Pacific coast of the
25 Tohoku Region, adjacent to the northeastern part of the Japanese mainland (Fig. 1). This "2011 Tohoku
26 Earthquake" is the strongest one known to have hit Japan. The earthquake triggered an extremely powerful
27 tsunami, causing many casualties (15,883 confirmed deaths, 2,656 missing, and 6,145 injured as of 9 August
28 2013, according to the Japanese National Police Agency) and severe destruction of infrastructure in the Tohoku
29 Region.

30 Many landslides were triggered by this earthquake in the wider Tohoku Region, with some concentrated in
31 the southern part of Fukushima Prefecture, in hilly residential areas of Miyagi and Fukushima Prefectures, and
32 on island areas of the Matsushima coast. Although the landslides were small for the magnitude of the earthquake
33 (compared to those landslides triggered by previous $M_j = 7-8$ earthquakes in the same area; Doshida and
34 Uchiyama, 2012), many showed very high mobility, with features typical of flowslide (fluid-like motion of
35 granular material; Bishop, 1973). The fluidized, rapid, long-travel movement of these landslides attracted
36 attention of geotechnical researchers and engineering geologists, because the landslides were scattered in
37 different areas, experienced little preceding precipitation, and displayed no evidence of occurrence beneath the
38 groundwater table. Understanding the initiation and movement mechanisms of these landslides is of great
39 importance to the mitigation of future coseismic geohazards. After the earthquake, we undertook a
40 reconnaissance field examination of the landslides, and investigated seven of them in detail. Here we describe
41 two landslides that occurred in the south part of Fukushima Prefecture and one in Tochigi Prefecture, and discuss
42 their possible initiation and movement mechanisms. By monitoring a landslide area during frequent aftershocks,
43 we could estimate the seismic response of the landslide area during the main shock which triggered the slope
44 instability. Through geotechnical tests on the shear behavior of the landslide materials for different moisture
45 contents, the post-failure behaviour of the landslide was inferred.

46

47 2. Study area

48 There were many landslides that occurred in southern Fukushima Prefecture and northern Tochigi Prefecture. As
49 samples of catastrophic landslides, we present data from two landslides (Hanokidaira and Shirasawa) from
50 Shirakawa City, and one occurring in the Oshino area (Fig. 1). Fig. 2 presents a geologic map of the study area,
51 which was based on the 1:200,000 Seamless Digital Geological Map of Japan (Geological Survey of Japan,
52 2003). This area mainly consists of Mesozoic granite, Paleozoic and Mesozoic sedimentary/ metamorphic rocks,
53 Neogene volcanic rocks and sedimentary rocks, and Quaternary fluvial deposits. All these three landslides
54 occurred in Pleistocene tephra and highly weathered welded tuff in hilly areas in the eastern part of the Ōu
55 Mountains. In the strata of the landslide areas, 12 layers of tephra resulting from eruptions of Nasu Volcano at
56 different times are recognized (Suzuki, 1992). In the following sections, geological details of each landslide are
57 introduced.

58 Fig. 3 shows the topography of the areas with the Hanokidaira (Fig. 3a), Shirasawa (Fig. 3b), and Oshino
59 (Fig. 3c) landslides. Fig.3a was based on the urban plan map (1/2500) of Shirakawa City, Fukushima Prefecture,
60 while Figs. 3b and 3c were taken from the digital maps of the Geospatial Information Authority of Japan,
61 because the urban plan maps in the scale of 1/2500 for these landslide areas are not available. The prefailure
62 slope of each landslide was steep, and recent fluvial erosion cut and steepened the slope toe (Chigira, 2011).

63

64 3. Methods

65 We measured their after-event topography using a laser rangefinder (TruPulse360^oB with a resolution of
66 0.1 m). We also used images from Google Earth to examine the topography of these landslides both before and
67 after the earthquake.

68 We used a portable seismometer (Character frequency: 1.9–2.1 Hz; Measurable amplitude of vibration: ± 2
69 mm; Sensitivity coefficient: 0.8 v kine^{-1}) to monitor earthquake aftershocks at the Hanokidaira landslide. This
70 seismometer has been developed for dense seismic observation, and has an extremely low power consumption (<

71 0.08 W). It records seismic movement continuously for six months using eight dry-cell batteries, at a sampling
72 rate of 250 Hz archiving one file per minute. We monitored aftershocks at the site during April 11–29, 2011 and
73 obtained many records.

74 We employed a series of intelligent ring-shear apparatuses developed at the Disaster Prevention Research
75 Institute, Kyoto University (Sassa et al., 2004), to examine the static and cyclic shear behaviors of samples that
76 were taken from near the sliding surfaces of the landslides. These samples were fully saturated or kept at their
77 natural moisture content during the shear tests, and were sheared under undrained conditions.

78
79

80 **4. Results**

81 *4.1 Features of fluidized landslides*

82 *4.1.1 Hanokidaira landslide*

83 The Hanokidaira landslide (Fig. 4a) originated on a southeast-facing slope of a hill located about 1 km north
84 of JR Shirakawa Station in Shirakawa City. Thirteen people were killed and 10 houses were destroyed by the
85 debris of the landslide. A small portion of the displaced material spilled into a golf driving range, forming a thin
86 deposit.

87 Stratigraphically from bottom to top, the slope consists of a basal deeply weathered welded tuff, a
88 palaeosol, weathered pumice and scoria partly with palaeosols, and forest soil. The bedded texture is
89 approximately parallel to the slope surface. The primary palaeosol, which overlies the welded tuff, outcropped,
90 and had striations on its surface (Fig. 4b). We dug into the deposits on the golf driving range and found that
91 grass was flattened and buried by the displaced materials, but not cut or displaced from its original place (Fig.
92 4c). Above the grass was a displaced ~10 cm thick palaeosol layer overlain by loose pumice. Many trees had
93 stood almost vertically while being transported. There were muddy striae on the left flank of the landslide (Fig.
94 4d) and also splashes of mud on the high position above the sliding surface (Fig. 4e), indicating that the
95 displaced mass experienced slide at first and then liquefaction, and the liquefied materials (mud) were splashed

96 during downslope movement. Therefore, the movement of this landslide can be classified as earth *slide-flow*,
97 according to Cruden and Varnes (1996).

98 Using the above-mentioned laser rangefinder, we surveyed the landslide area to prepare a longitudinal
99 section (Fig. 4f). The main part of the displaced materials spread into a residential area at the foot of the hill,
100 exhibiting substantial fluid-like flow. The apparent mobilized friction angle of the slide was about 9.6° ,
101 measured from point I to I' in Fig. 4a. According to the measured topography, we estimated that the volume of
102 the displaced material was about $100,000 \text{ m}^3$, with a horizontal travel distance of about 290 m and a total relief
103 of about 50 m.

104 No standing ground water or groundwater seepage was found in the landslide source area. However, local
105 residents told us that there was a continuous spring at the toe part of the slope. Therefore, we inferred that
106 standing ground water existed in the slope before the occurrence of the landslide. Due to stabilization
107 countermeasures, the debris on the lower part of the slope, near point T in Fig. 4a, has been removed and the
108 slope continues to yield water from small springs (observation in July 2012).

109 At the Hanokidaira landslide source area, tephra layers exposed during excavation for stabilization. The
110 existence of palaeo-valley had been identified (Fig. 4g). The upper surface of the welded tuff (and the palaeosol)
111 indicates a substantially dissected hilly area with a palaeo-valley aligned approximately east–west and sloping to
112 the east (PV-M in Fig. 4g). The palaeo-valley had a tributary (side valley) sloping up towards the present highest
113 point in the local landscape (PV-T in Fig. 4g). The most prominent palaeosol in the sequence is developed on
114 this hilly palaeo-surface, which was subsequently mantled and largely buried by thick layers of air-fall volcanic
115 ash. Through erosional reworking of the tephra during the long interval during which all of the layers were
116 deposited, the thickness of the ash pile along the axis of the palaeo-valley has become greater than that on the
117 flanks of the palaeo-valley. The landslide has substantially cleaned out the tephra down to the palaeosol
118 developed on the weathered welded tuff.

119

120 4.1.2. Shirasawa landslide

121 The Shirasawa landslide (Fig. 5) is located approximately 4 km northeast of the Hanokidaira landslide. It
122 originated from a gully head. Like the Hanokidaira landslide, the soil layers in the source area, from the bottom
123 to the ground surface, are a palaeosol, pumice, and weathered volcanic ash with pumice. The bedded texture
124 approximately parallels to the slope surface. The displaced materials slid from the source area above the
125 palaeosol (Fig. 5b), and the left flank was also imprinted by muddy striae and splashes of mud on a high position
126 above the sliding surface (Fig. 5c). Therefore, the movement can be classified as earth *slide-flow* like the
127 Hanokidaira landslide. The landslide began on a 13° slope, slid onto a slope of 46°, then turned south and
128 travelled 230 m along the valley. The total flow length of the slide was 331 m, with a relief of 70 m, giving an
129 apparent mobilized friction angle of 11.9°. The source area was 95 m long, 30 m wide and 3 m thick on average,
130 giving a landslide volume of about 9000 m³. This landslide occurred in a non-residential area and no casualties
131 or injuries were reported.

132

133 4.1.3. Oshino landslide

134 The Oshino landslide (Fig. 6a) is located 45 km south of Shirakawa City. The landslide originated on a
135 northeast- to north-facing slope, below which are terraced paddy fields. The source area is 45 m long, 55 m wide
136 and 4 m thick on average; therefore the landslide volume was approximately 10,000 m³. The displaced materials
137 traveled about 100 m across a rice paddy, and then descended a 10 m steep step and continued for further 65 m
138 across a lower rice paddy (Fig. 6d). The apparent mobilized friction angle of this landslide was 9.5°. The
139 deposits had a thickness of about 3 m in both paddies. Many trees were standing vertically on the landslide toe.
140 We dug a pit at the toe and found that the rice paddy soil had been barely disturbed.

141 The soil layers of the source area were a palaeosol, weathered volcanic ash, and pumice respectively from
142 the base of the sliding surface to the top. Fig. 6b presents the outcropped sliding surface with palaeosol

143 underlain. The bedded texture approximately parallels to the slope surface. On the left flank, muddy striae and
144 splashes of mud on the high position above the sliding surface were also observed (Fig. 6c), indicating that this
145 landslide can be classified as earth *slide-flow* according to the movement of displaced mass. A witness reported
146 that the sliding occurred and stopped in some seconds during the strong seismic motion.

147

148 4.2. Seismic motion

149 Japan has a dense network of seismic stations including two permanent strong-motion stations in Shirakawa
150 City: a Kyoshin Network (K-NET Shirakawa) station of the National Research Institute for Earth Science and
151 Disaster Prevention (NIED), Japan, and a station operated by the Japan Meteorology Agency (JMA
152 Shirakawa-shi Kakunai station). The seismic waves recorded by these two stations during the main shock are
153 presented in Fig. 7a-f, respectively. At both stations, the vertical component of motion was the smallest, and N-S
154 component the greatest. Nevertheless, the peak values were different. The K-NET station had a peak acceleration
155 of 1295.2 gal, while the JMA station recorded a maximum of 371.5gal, about one-third of the former. The
156 K-NET station is about 2.8 km southwest of the Hanokidaira landslide, while the JMA station is about 800 m
157 south. Therefore, the landslide may have experienced a similar level of a seismic response as that at the JMA
158 station during the main shock. However, seismic responses of a site depend greatly on the local characteristics of
159 soil/rock layers and topography. To better understand the seismic motion of the Hanokidaira area, we installed a
160 seismometer close to the top of the landslide area on April 11, 2011 to record aftershocks and estimated the
161 seismic response during the main shock (Fig. 7g-i) by means of the site-effect substitution method (Hata et al.,
162 2011, 2012). The estimated seismic wave had a peak acceleration of about 907 gal in an N-S direction and 778
163 gal in an E-W direction, about 2.4 and 2.3 times greater than the records from the JMA station respectively.
164 Although the estimated vertical acceleration had a smaller peak value of 463 gal, this value is about 3.4 times
165 greater than that recorded at the JMA station, indicating that the Hanokidaira landslide area had a stronger site

166 effect on the vertical motion. From Fig. 7g-i, we infer that the peak seismic motion on the slope at Hanokidaira
167 was greater than 1 g, and triggered the failure during the main shock.

168 *4.3. Geotechnical aspect*

169 *4.3.1. Setup of shear test*

170 Field surveys revealed that all these landslides had a palaeosol as their basal layer, and all the materials
171 above this layer had been displaced. This suggests that the boundary between the palaeosol layer and the
172 overlying pumice layer played a key role in the initiation and movement of the landslides, and the shear failure
173 formed a sliding surface in both soil layers. Although our field surveys failed to find standing water, i.e., the soil
174 layers might not have been in a fully saturated state, we found that the field moisture content of these soil layers
175 was very high. Also the landslides were all triggered on slopes with small valleys below where ground water
176 tends to converge. We infer that the soil layers near the sliding surface had a very high moisture content or been
177 nearly saturated, and hence liquefaction led to the high mobility of these landslides.

178 In 2011, disturbed samples of more than 40 kg were taken from the palaeosol (S1) and pumice with scoria
179 (S2) in the source area of each landslide for geotechnical tests. In July 2012, we also took intact samples from
180 the palaeosol layer of the Hanokidaira landslide exposed during stabilization work (S1' in Fig. 4). Two blocks
181 (about 40×40×30 cm for each) were taken and transported in plastic bags to keep moisture content.

182 At first, we performed direct shear box tests on intact and remolded samples of S1 taken from the
183 Hanokidaira landslide to examine the shear strength of S1 in intact state and also to examine the possible
184 difference between the intact and remolded samples. We also performed undrained shear tests on the remolded
185 samples of S1 and S2 taken from all these landslides at natural moisture content or fully saturated state, and
186 found that they all can be fully liquefied. In the following, we present the results from tests on the samples taken
187 from the Hanokidaira landslide as an example. S1 from the Hanokidaira landslide had a specific gravity of

188 approximately 2.70 g cm^{-3} , dry density of $0.48\text{--}0.66 \text{ g cm}^{-3}$, natural moisture content of 94–160% and void ratio
189 of 3.10–4.18, while sample S2 had a specific gravity of 2.68 g cm^{-3} , dry density of $0.51\text{--}0.64 \text{ g cm}^{-3}$, natural
190 moisture content of 94–151%, and void ratio of 3.25–4.25. The intact blocks of S1 had a natural moisture
191 content of 114% and dry density of 0.636 g cm^{-3} .

192 Because the tests were carried out to examine the shear behavior of the soil in the source area, test
193 conditions were designed to represent a soil element on the sliding surface overlain by an 8 m thick soil
194 (measured vertically) for the Hanokidaira landslide. The normal and shear stresses on the sliding surface were
195 calculated using a slope angle of 16° (Fig. 4f), and a unit weight of 11.4 kN m^{-3} , which was calculated from the
196 measured in situ density of the sample, assuming that no groundwater table existed in the slide. Thus, the
197 estimated normal stress (σ_i) was 84 kPa and shear stress (τ_i) was 24 kPa.

198 We performed direct shear box tests on the intact or remolded sample of S1 under consolidated drained
199 conditions following the standard of ASTM D3080. The shear box, assembled with the top and bottom halves of
200 the box screwed, has a size of $10\times 10\times 10 \text{ cm}$. The remolded samples were prepared to be approximately the same
201 initial dry density as the intact one. We sheared the sample at natural moisture content and also at fully saturated
202 state. For saturated tests, we put the shear box with sample into distilled water for 4 days to ensure full
203 saturation.

204 We performed undrained ring shear tests on saturated samples following these steps: (1) putting remodeled
205 S1, directly taken from the in-tack blocks in natural water content without any dry process, or air dried S2 into
206 the shear box; (2) saturating the samples using CO_2 and de-aired water; (3) normally consolidating the sample
207 under a given stress state; (4) undrained shearing the sample by applying cyclic loadings or increasing only shear
208 stress (static shearing). For tests on samples with the natural moisture content, we put the remolded natural
209 samples into the shear box and then went through steps (3) and (4). More detailed procedures for performing
210 undrained shear tests in a ring shear apparatus are described by Sassa et al (2003).

211 We also performed undrained shear tests on fully saturated air-dried or oven-dried samples of S1 to
212 examine the possible effect of drying process on its undrained shear behavior. As pointed out by Chigira et al
213 (2012), the dominant clay mineral in the palaeosol of the studied landslides is halloysite, which is normally
214 formed by hydrothermal alteration of volcanic rocks (Kerr, 1952); while air or oven drying would result in the
215 dehydration of interlayer water of halloysite (Wesley, 1973, 1977; Okada and Ossaka, 1983). Nevertheless,
216 detailed examination on the possible effects of drying process on palaeosol properties is beyond the focus of this
217 study; here we only present the results of tests on those samples with natural moisture contents.

218

219 *4.3.2. Results of direct shear box tests on samples S1*

220 We performed direct shear box tests under differing normal stress levels (84, 64 and 41 kPa, respectively).
221 The intact samples were kept at natural moisture contents or fully saturated, while the remolded samples were
222 kept at their natural moisture contents. The representative failure envelopes from these tests are shown in Fig. 8.
223 The intact sample at natural moisture content presented an angle of shearing resistance of $\varphi' = 27^\circ$ with an
224 apparent cohesion, c , of 22 kPa (Fig. 8). These strength parameters varied greatly when the sample was
225 remolded, showing $\varphi' = 14^\circ$ and $c = 26$ kPa. Fully saturated intact sample behaved slight change in the strength
226 parameters, showing $\varphi' = 26^\circ$ and $c = 19$ kPa, probably because the natural moisture content was originally very
227 high (the saturation degree was 94.3%).

228 Consolidated drained triaxial compression tests were also conducted on the intact and remolded samples of
229 S2 taken from the location shown in Fig. 4, and the results showed that saturated intact S2 has $\varphi' = 33.0^\circ$ and $c =$
230 37.4, while the saturated remolded S2 has $\varphi' = 34.2^\circ$ and $c = 2.7$ (K. Sasahara, personal communication, January
231 12, 2012). The failure envelopes for intact and remolded samples of S2 were compared to these of S1 in Fig. 8.
232 Intact S2 has a greater shear strength than intact S1 at any given normal stress.

233

234 4.3.3. Results of cyclic shearing at natural moisture contents

235 Undrained cyclic shear tests were conducted on both samples at natural moisture contents (114% and 140%
236 for S1 and S2, respectively). After being placed in the shear box and normally consolidated, the samples were
237 subjected to a cyclic shear stress, with an amplitude of 24 kPa (i.e., 24 kPa cyclic load superimposed onto the 24
238 kPa gravitational shear load) and a frequency of 0.5 Hz under undrained conditions. We used this low-frequency
239 for better monitoring the response of pore-water pressure that might be generated during the shearing. Fig. 9
240 presents the results as time series data for both samples. In these tests, normal stress was kept constant. However,
241 the cyclic shear stress was not faithfully applied by the control system, probably because this ring shear
242 apparatus was designed for high stress level (maximal normal stress: 2 MPa), and the cyclic shear stress of 24
243 kPa is relatively too small. Nevertheless, in both tests, shear failure was triggered due to the introduction of
244 cyclic shear loading, and continued even after the cyclic shear loading ceased. With continuing cyclic shearing,
245 the shear resistance became smaller than the initial shear stress. This enabled continuing shear failure and shear
246 resistance lowered further to approximately zero.

247 The monitored pore-water pressure was small, likely due to the air left inside the samples and also inside
248 the pore-water pressure measuring system. The highly compressible air would cause a delayed reaction to the
249 change in pore-water pressure. The low permeability of the sample (clayey soil) may also delay the response of
250 the pore-water pressure monitoring system, because the excess pore water pressure was generated within the
251 shear zone, while the pore-water pressure measuring system was installed near but outside of the shear zone
252 (Wang et al, 2007). Disturbed sample S2 has a permeability of approximate $2.9 \times 10^{-2} \text{ cm s}^{-1}$ at a dry density of
253 about 0.51 g cm^{-3} , while sample S1 has a permeability of about $1.3 \times 10^{-5} \text{ cm s}^{-1}$ at a dry density of about 0.64 g
254 cm^{-3} , smaller than that of S2. This may be the reason why the monitored excess pore water pressure in Fig. 9a
255 was smaller than that in Fig. 9b. However, we inferred that the excess pore-water pressure built up within the
256 shear zone had reached to a value approximately equal to the total normal stress, such that the shear resistance
257 lowered approximately to zero after 40 seconds in both tests.

258 Although additional shear stress of about 24 kPa had been applied to the initial shear stress once (Fig. 9a),
259 the applied cyclic shear stress was smaller than 24 kPa in general, resulting in changes in the number of cycles
260 necessary for the initiation of shear failure, but has no effect on the post failure shear behavior (Wang, 2000).
261 Therefore, considering the strong motion and long duration of the main shock of the earthquake and the inferred
262 degree of saturation, we concluded that liquefaction occurred within the soil layers near the sliding surface.

263
264

265 *4.3.4. Results of static shearing at natural moisture content*

266

267 Considering that the side wall of the landslide mass would suffer from static shearing after the landsliding
268 was initiated by the earthquake, we performed static shearing test on sample S2. In the shear box, the sample
269 with a moisture content of 140% was normally consolidated under the normal stress of 84 kPa and shear stress of
270 24 kPa, and then sheared by increasing shear stress monotonically under undrained condition. Fig. 10 shows an
271 example of the results of this test. With increasing shear stress, excess pore water pressure was built up
272 gradually. After shear failure occurred, shear resistance gradually lowered to a very small value close to zero.
273 Vertical displacement (positive with sample consolidation) also occurred as shearing progressed.

274 After the test, we opened the drainage system, and found water being expelled from the shear box.
275 Therefore, even though the sample was not fully saturated, the abundant water in the sample allowed generation
276 of high pore-water pressure after shear failure was triggered.

277

278 *4.3.5. Shear rate dependency of the palaeosol*

279 Once shear failure occurred, we expected that the displaced landslide material might not accelerate at a
280 constant rate due to strength change with shear rate. To examine this effect, we sheared both samples at different
281 shear rates. The samples were first saturated, and consolidated under a total normal stress of about 200 kPa. This
282 normal stress was chosen for a better performance of the control system and comparisons with the results of our
283 previous tests under the same condition. Then the sample was sheared to a residual state using a

284 shear-speed-controlled method. We performed multistage test, which has been found to produce results similar
285 to the test of individual samples (Bromhead, 1992; Tika et al., 1996; Tiwari and Marui, 2004; Suzuki et al.,
286 2004; Wang et al., 2010). After the measurement of residual shear strength at a given shear rate, we repeatedly
287 changed the shear rate and then measured the residual shear strength at this differing shear rate. Through this
288 method, the residual shear strengths at different shear rates were measured (Fig. 11). We found that the effect of
289 shear rate on the residual shear strength was significant for the palaeosol sample, and that the shear strength of
290 sample S2 was less affected by the shear rate. The residual shear strength was greater than 150 kPa when the
291 shear rate was smaller than 0.6 mm s^{-1} , but was smaller than 50 kPa when the shear rate was greater than 1 mm
292 s^{-1} . Although the reason for this sharp decrease in shear resistance with shear rate is unclear, it can be concluded
293 that the shear resistance became very small when the displaced material moved faster.

294

295 5. Discussion

296 To analyze landside mobility, a parameter of travel angle (φ_a), sometimes called apparent friction angle, has
297 been widely used (Scheidegger, 1973; Cruden and Varnes, 1996). The parameter is defined as $\tan\varphi_a = H/L$,
298 where H is the landslide height (the difference in elevation between the crown and the tip of the landslide), and L
299 is the horizontal distance of the landslide from the toe to the head scarp. Small values of φ_a mean high mobility
300 and the value tends to decrease with increasing volume (Scheidegger, 1973; Voight et al., 1983; Legros, 2001;
301 Crosta et al., 2005). In addition, landslides involving volcanic materials usually have higher mobility (Hayashi
302 and Self, 1992; Legros, 2001). Recently studies showed that some small, rapid, fluidized landslides due to
303 rainfall (Wang, 2000) or earthquakes (Crosta et al., 2005) have φ_a of about 10° . The landslides we investigated
304 (Figs. 4 to 6) also had similar φ_a values of 9.6° (Hanokidaira), 11.9° (Shirasawa), 11.8° (Oshino), and 9.5° (Slide
305 I in Fig. 6), showing high mobility, although most of the landslide materials were unsaturated.

306 Iverson et al. (1997) pointed out that liquefaction plays a key role in the formation of fluidized landslides. In
307 the case of coseismic landslides, the slope instability may result from 1) coseismic force that may make

308 downslope shear stress greater than shear strength, and (2) buildup of excess pore-water pressure within fully or
309 partially saturated soil layers that may lower the shear strength. In the case of fluidized coseismic landslides, if
310 the excess pore-water pressure increases and the effective normal stress decreases, shear failure may occur, and
311 excess pore-water pressure may further increase with the progress of the failure. As shown in Fig. 8, sample S2
312 has lower shear resistance than sample S1, indicating that if coseismic force is great enough, the first shear
313 failure occurs within the palaeosol layer. During the subsequent downslope movement of the displaced material,
314 the overlying soil layer of pumice and scoria experiences further vibration and static shearing, leading to
315 liquefaction. This may be the reason why the deposited landslide materials mainly consist of fluidized pumice
316 with little palaeosol. Therefore, for the risk analysis of fluidized landslides, it is necessary to better understand
317 both the coseismic shear and static shear behaviors of soil layers within a slope.

318 Numerous tests have been performed on fully saturated sands to examine their liquefaction behavior (c.f.,
319 Seed, 1966; Finn, 1981; Ishihara, 1993). However, recent studies reveal that even under unsaturated conditions,
320 sand with a highly compressible soil may liquefy (Kazama et al., 2006; Unno et al, 2006, 2008). The undrained
321 shear tests on samples S1 and S2 (Figs. 9 and 10) indicate that full liquefaction can occur in both samples after
322 shear failure although they are only partially saturated.

323 The amplification of seismic movement due to slope topography has been recognized (Davis and West,
324 1973; Ashford et al., 1997; Nishimura and Morii, 1984; Kurita et al., 2003; Buech et al., 2010). Although greater
325 amplification normally occurs on the crest of a slope, the vertical motion of a bedrock-dominated slope is rarely
326 affected by topography (Kurita et al., 2003). Recent studies reveal that the seismic motion especially in the
327 vertical direction is also dependent on the degree of soil saturation resulting in non-linear soil response (Yang
328 and Sato, 2000; Tobita et al, 2010). The estimated seismic motion shown in Fig. 7g-i was based on a practical
329 estimation method of strong ground motion (called “site-effect substitution method”) that was proposed by Hata
330 et al (2011). This method uses the records of aftershocks both at the site of interest and at a nearby permanent
331 strong motion observation station, and the record during the main shock at the observation station. Because this

332 method takes account of not only the difference of site amplification factors, but also the difference of site phase
333 effects between the site of interest and the observation station, it can estimate time histories of strong ground
334 motion at the site of interest with high accuracy. However, this method does not consider the effect of strong
335 nonlinearity of shallow soft soil, which can greatly change the ground-motion amplification of shallow soil
336 layers (Field, et al., 1997; Beresnev et al., 1998). Therefore, the estimated seismic motion might be
337 representative of the response of engineering bedrock, as considered in the field of earthquake engineering,
338 rather than of shallow soil. The engineering bedrock in earthquake engineering is normally determined by the
339 S-wave velocity (V_s) of the bedrock, approximately 300–700 m s⁻¹. The seismic motion of surficial soil with
340 smaller V_s tends to be greater. Further, soil layers in the source area had high moisture content but were not fully
341 saturated. This state could result in a large amplification of vertical motion. We checked the seismic recordings
342 obtained during the aftershocks and found that the vertical motion on the landslide site could be up to about 9
343 times that recorded by the nearby JMA station, although the estimated vertical motion during the main shock
344 (Fig. 7i) is about 3.4 times that of the JMA records. Although we did not perform further analysis on the possible
345 elastic-plastic earthquake response of the slope, the ground motion of the slope during the main shock may have
346 been much stronger than the estimates in Fig. 7g-i.

347 In the analysis of coseismic landsliding, the effect of vertical motion on the instability of slopes has been
348 largely ignored. Nevertheless, recent field monitoring and theoretical analysis have revealed that vertical motion
349 also plays an important role in landslide initiation (Yang, 2007). For the studied landslides, strong vertical
350 motion may have contributed to their initiation through two processes: coseismic shear force and compression
351 (or collapse) of unsaturated soil layers, which build up excess pore-water pressure in unsaturated soils during the
352 earthquake. Although the seismic response of a given site to a given earthquake depends on many factors, the
353 direct seismic observation at a landslide site may provide promising data for better understanding the initiation
354 of coseismic landslides.

355

356 **6. Conclusions**

357 During the 2011 Tohoku Earthquake off the Pacific coast of Japan, many landslides were triggered in urban
358 residential regions and on natural slopes, killing more than a dozen people and destroying many homes. These
359 landslides were not very large but characterized by high mobility and a long travel distance. Through field
360 surveys, analyses of seismic motion, and experimental geotechnical examination, their possible initiation and
361 movement mechanisms were examined. The conclusions are as follows.

362 (1) The landslides in pyroclastic fall deposits showed very high mobility, with the occurrence of
363 liquefaction. Valley incision was distinct at the toe of these landslides, indicating that the upper slope of the
364 landslide source area was less stable state before the earthquake.

365 (2) Although the epicenter was distant, and the motion recorded by a nearby permanent seismic station was
366 not strong, the seismic motion at the Hanokidaira landslide area was very strong due to local site effects.

367 (3) For each landslide, the sliding surface was along the boundary between a palaeosol developed on
368 welded tuff and the overlying pyroclastic fall deposits (pumice and scoria). Shear failure might have occurred at
369 first on the palaeosol layer, but liquefaction might have triggered in both the palaeosol layer and the overlying
370 pumice layer, resulting in the high mobility of displaced landslide materials. These soil layers are highly
371 liquefiable in saturated condition when subjected to cyclic shear loading. High pore-water pressure can also be
372 generated even when the soil layers are not fully saturated.

373 (4) The shear resistance of the basal palaeosol markedly decreased with increasing shear rate, indicating
374 that in the case of a shear failure within the palaeosol, the shear resistance of the sliding surface might become
375 smaller with an increase of sliding velocity, thus allowing accelerating movement of the displaced landslide
376 materials.

377

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388

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479 Captions:

480 **Fig. 1.** Locations of the epicenter of the 2011 Tohoku earthquake off the Pacific coast of Japan and the landslides
481 presented in this study (after Google Map). ○: Location of city; ●: Location of landslide.

482

483 **Fig. 2.** Geological map of the study area (after Geological Survey of Japan, 2003)

484

485 **Fig. 3.** Topographic maps. (a) Hanokidaira landslide area (based on the planning map of Shirakawa City with a
486 contour interval of 2 m); (b) and (c): Shirasawa and Oshino areas, respectively (based on the digital map of
487 Geospatial Information Authority of Japan with a contour interval of 10 m).

488

489 **Fig. 4.** Hanokidaira landslide. (a) Oblique view (after Takeshi 2011); (b) view of the left side margin and sliding
490 surface; (c) buried grass on the golf driving range and palaeosol layer above the grass; (d) muddy striae on the
491 left-side margin; (e) splashed mud on left-side margin; (f) longitudinal section along line I-I'; (g) outcropped
492 palaeo-valley in the source area of the Hanokidaira landslide. PV-M: main palaeo-valley; PV-T: tributary of the
493 main palaeo-valley. S1 and S1': Sampling locations for disturbed and intact samples of palaeosol, respectively.

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495 **Fig. 5.** Shirasawa landslide. (a) Oblique view (after Takeshi 2011); (b) sliding surface and outcropping palaeosol
496 on the source area; (c) Muddy striae and splashed mud on the left side margin.

497

498 **Fig. 6.** Oshino landslide. (a) Oblique view, (b) Outcropped sliding surface and palaeosol; (c) muddy striae and
499 splashed mud on left side of the margin; (d) longitudinal section along I-I'.

500

501 **Fig. 7.** Seismic records and maximum acceleration from the K-NET Shirakawa station (a-c), the JMA
502 Shirakawa-shi Kakunai station (d-f), and the estimated seismic waves in the Hanokidaira area (g-i). EW, NS,
503 UD: seismic motion in the direction of east–west, north–south, and up–down (vertical).

504

505 **Fig. 8.** Drained shear strength and failure envelopes of the intact and remolded sample S1 at natural moisture
506 content by direct shear box tests. Also showing the failure envelopes of the intact and remolded sample S2 at
507 saturated state obtained by triaxial compression tests.

508

509 **Fig. 9.** Undrained cyclic shear tests on saturated samples S1 (a) and S2 (b). The dry densities (after
510 consolidation) were 0.70 and 0.51 g cm⁻³ for S1 and S2, respectively.

511

512 **Fig. 10.** Undrained shear test on sample S2 (moisture content: 140%). (a) Time series data of stresses, pressure
513 and shear displacement; (b) vertical displacement (positive indicates sample consolidation). The dry density of
514 the sample after consolidation was 0.55 g cm⁻³.

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516 **Fig. 11.** Residual shear resistance of saturated S1 at different shear rates.

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539 Figures

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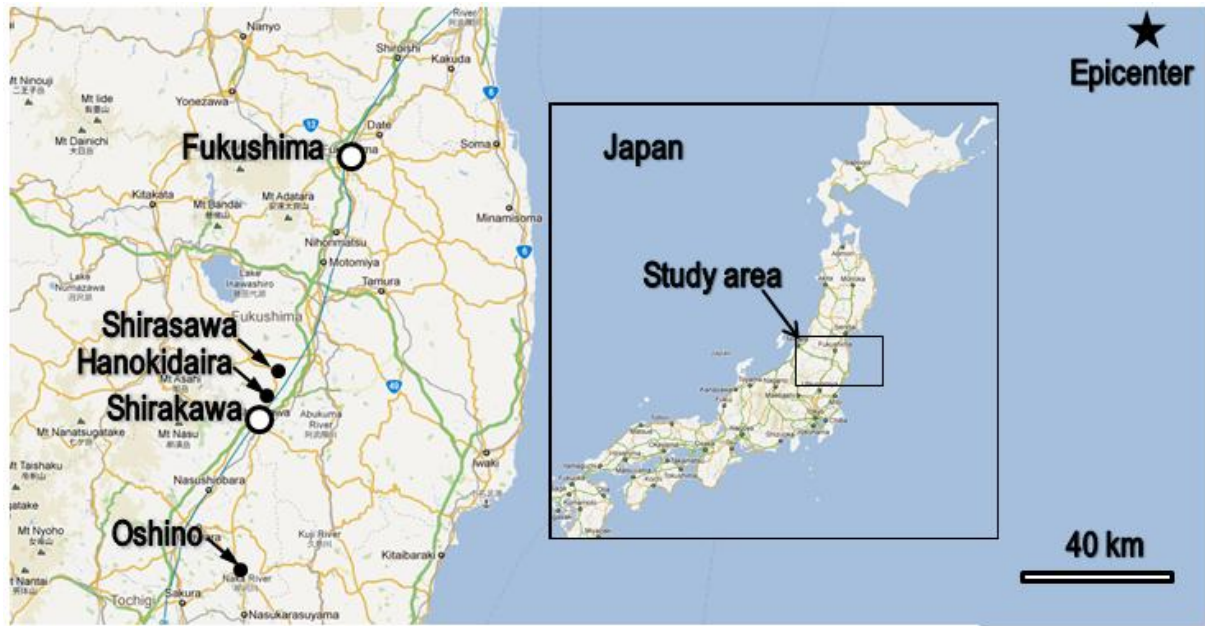
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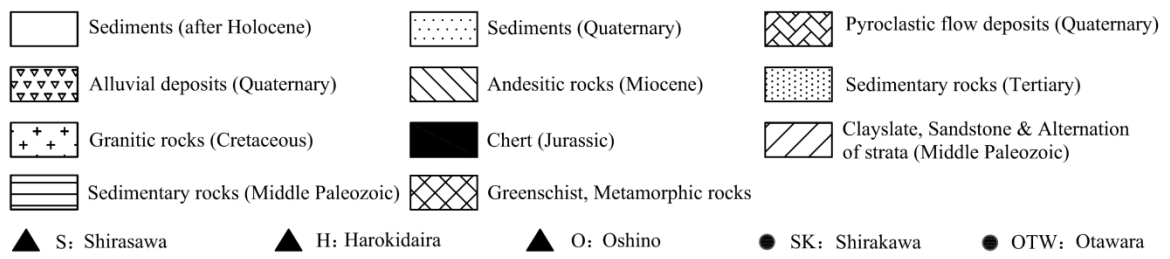


Fig. 2 Geological map of study area

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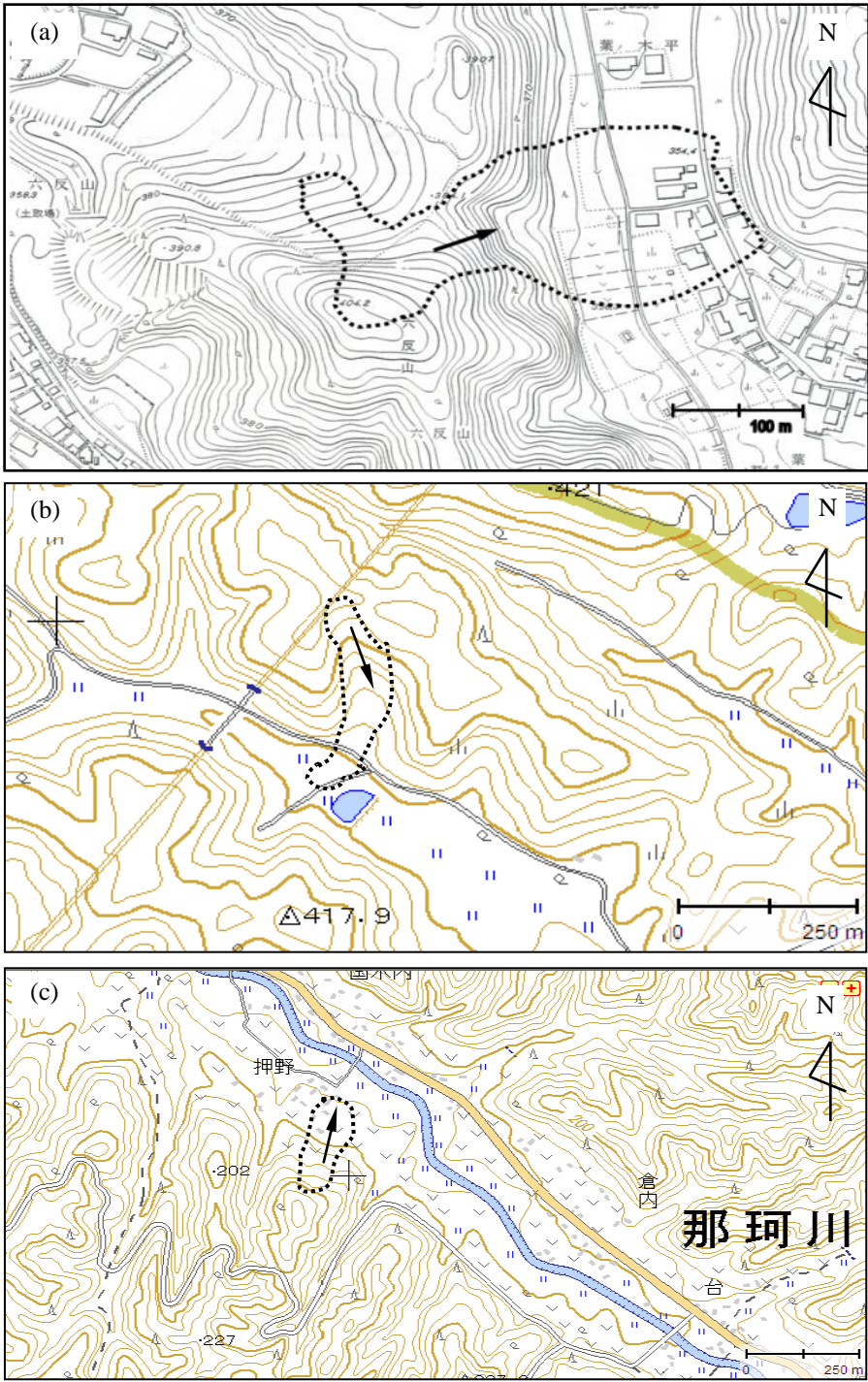


Fig. 3.

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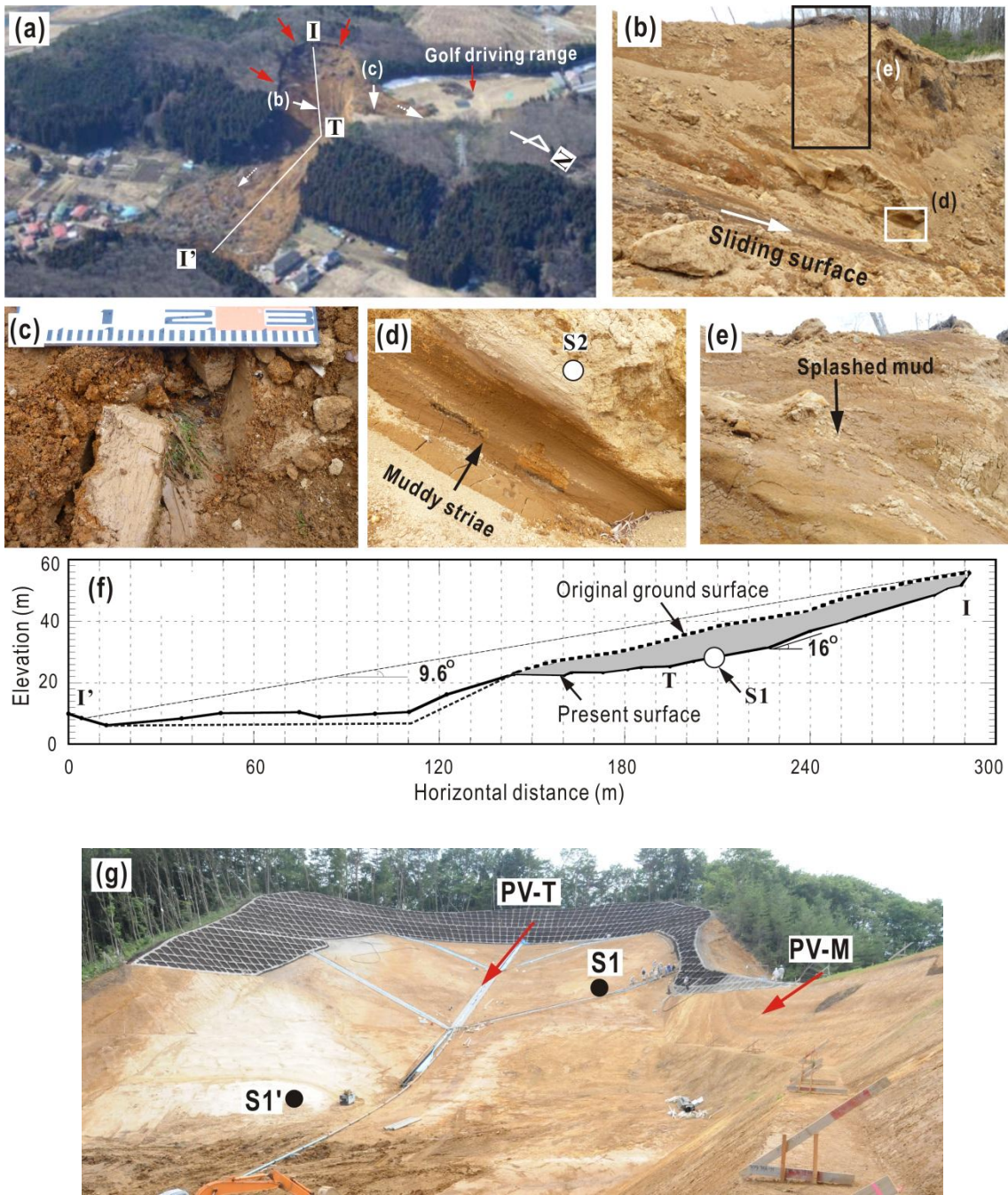


Fig. 4.

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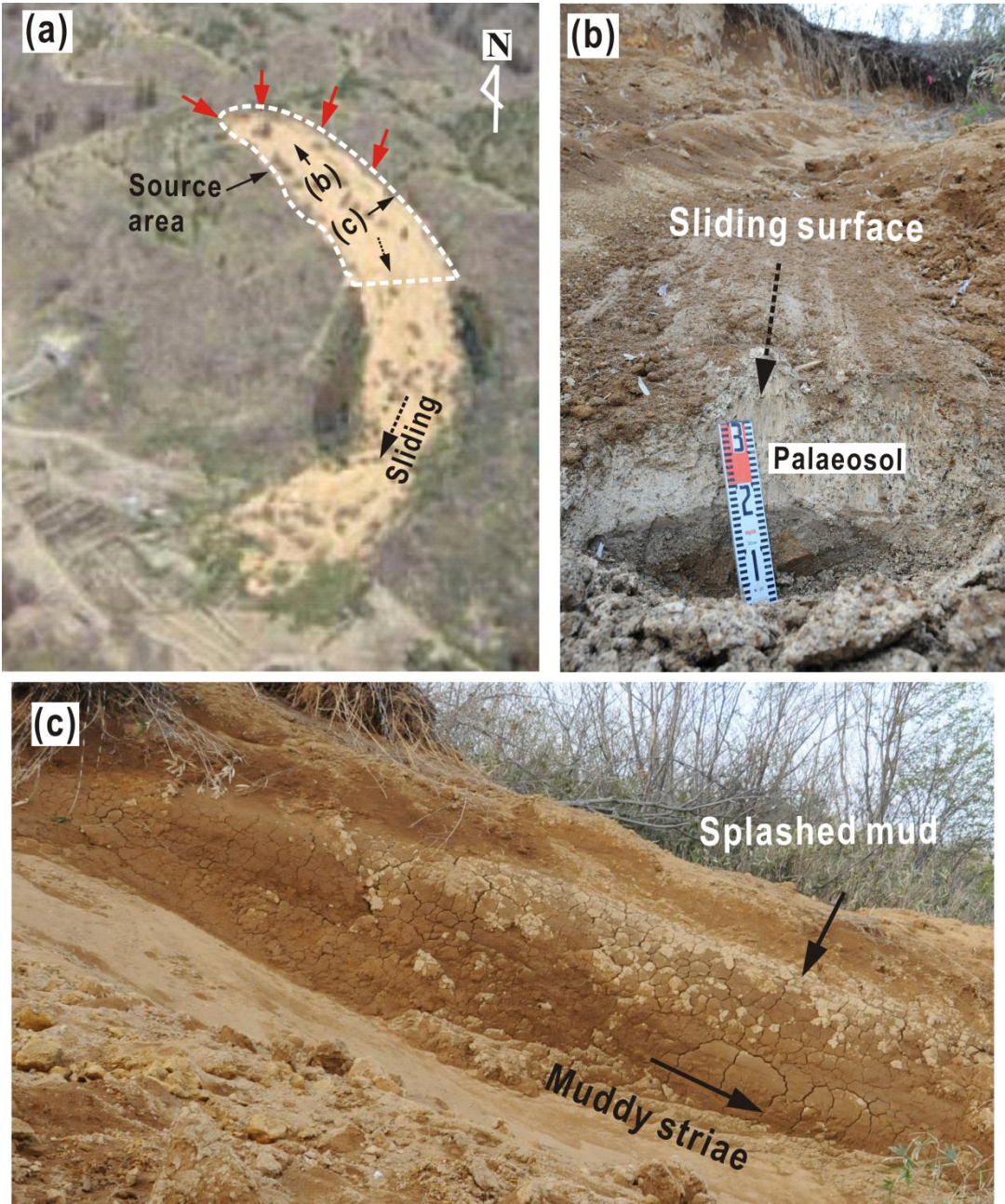


Fig. 5.

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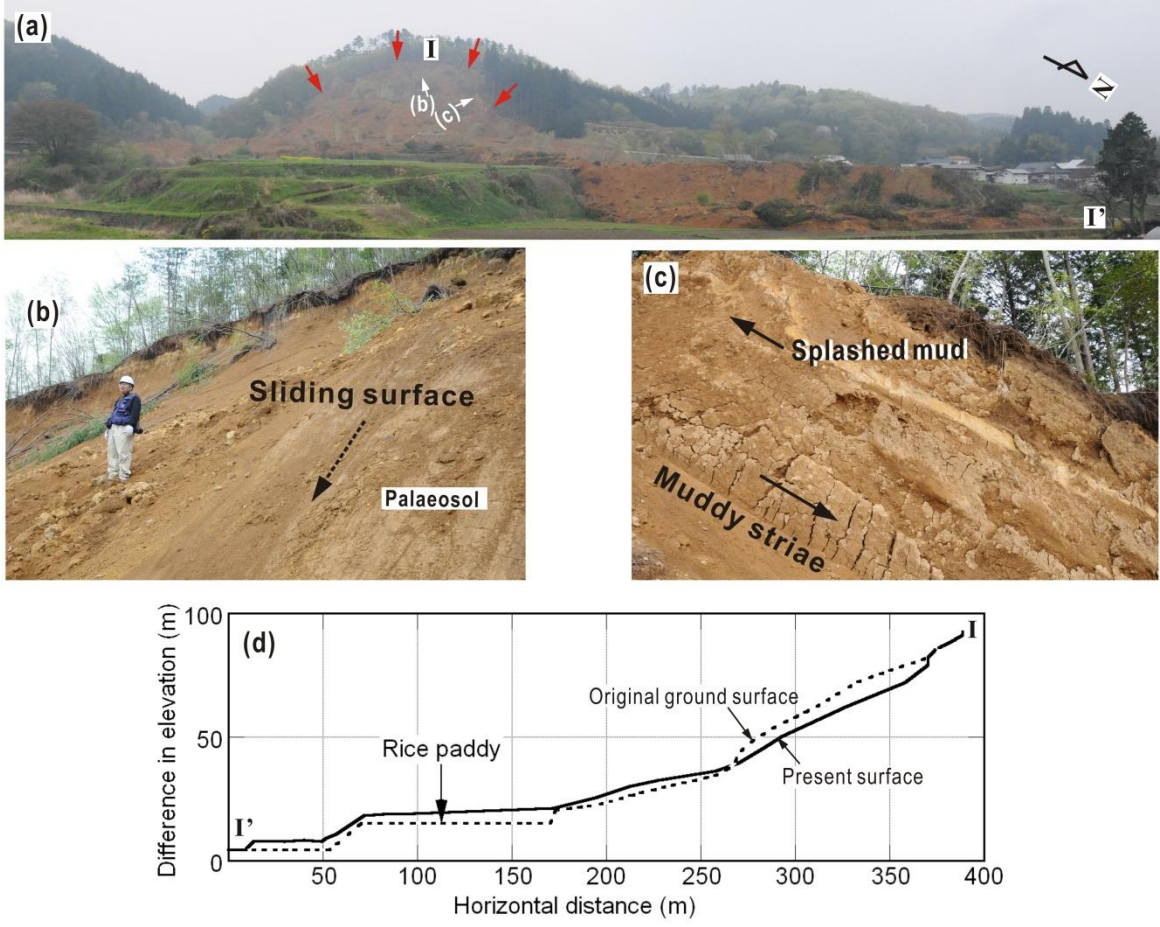


Fig. 6.

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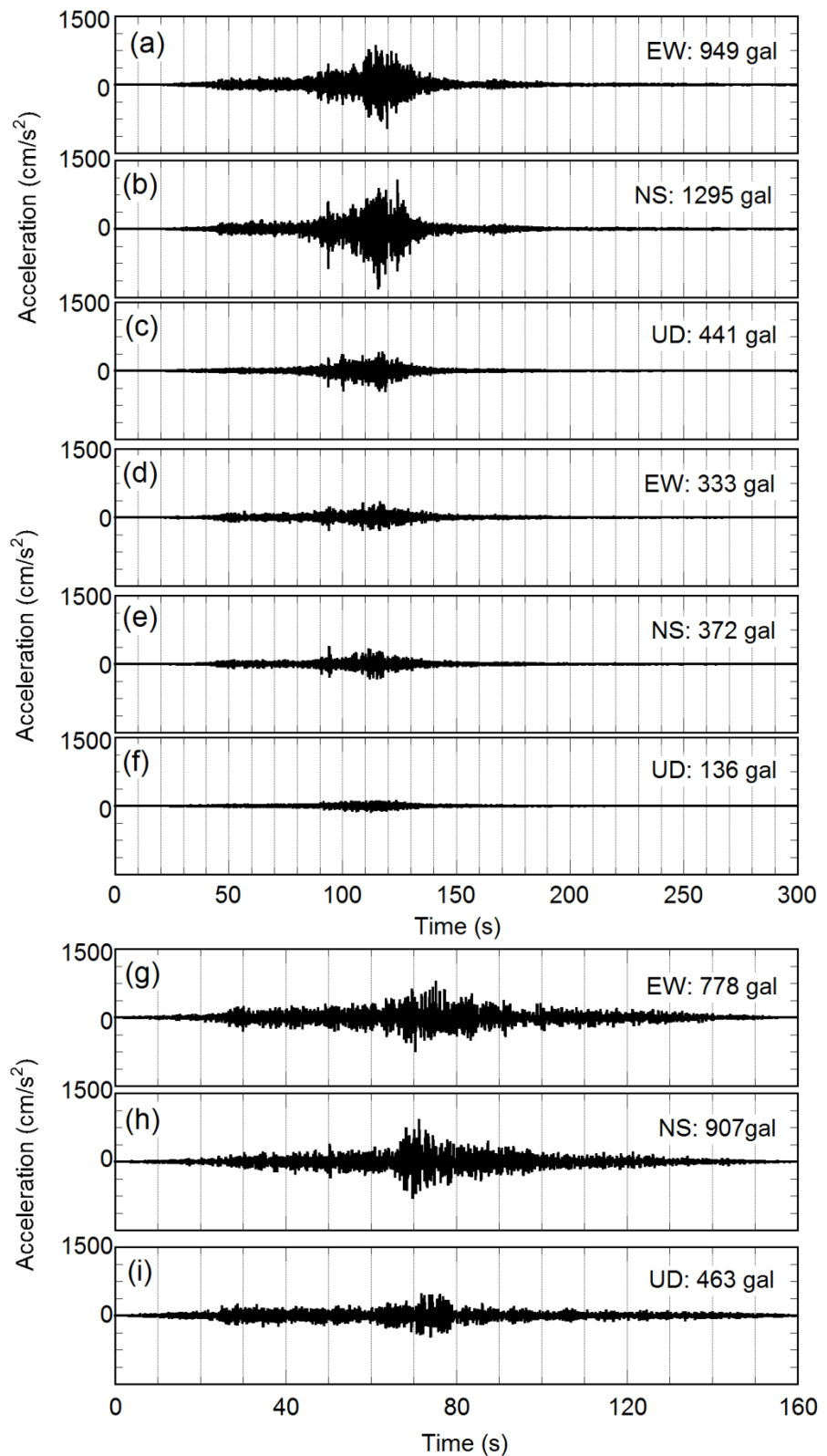
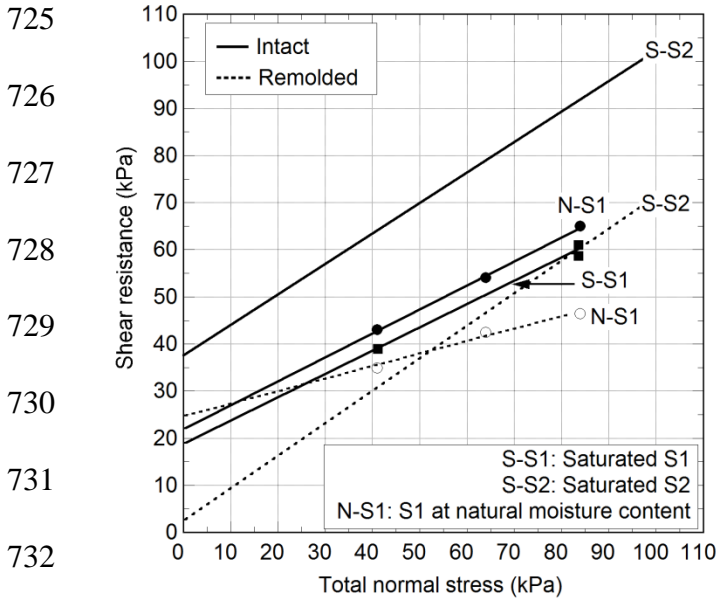
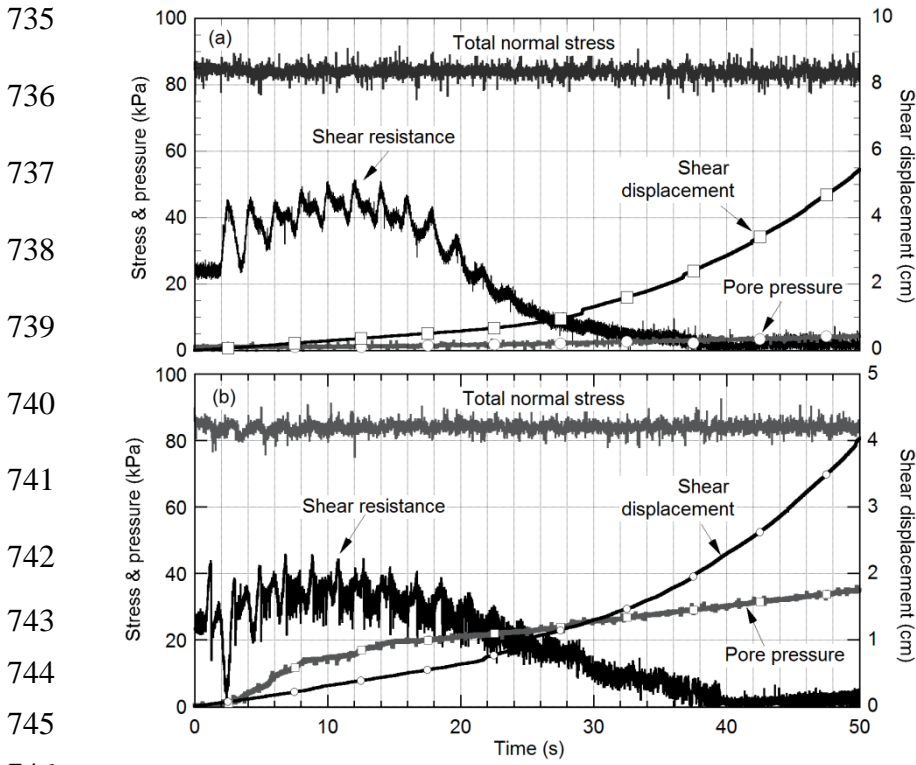


Fig. 7.



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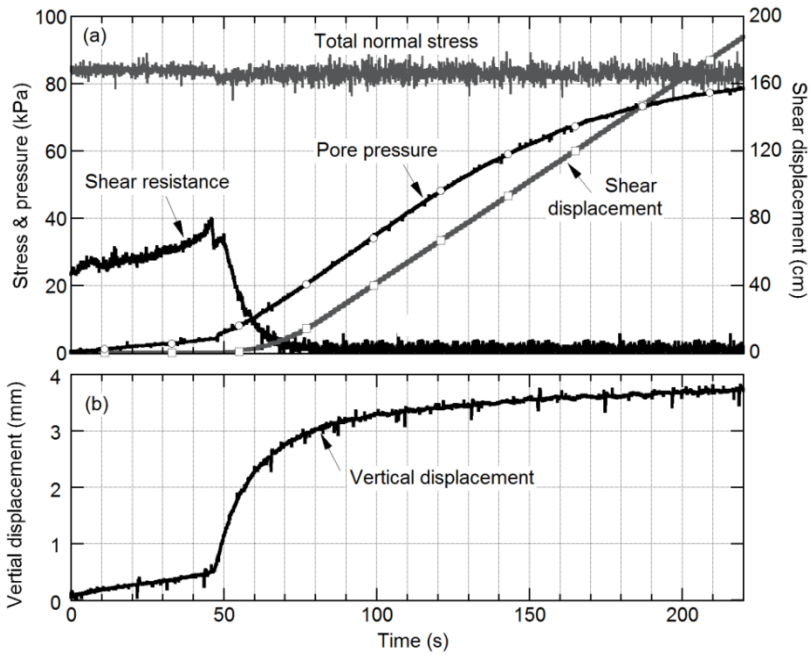
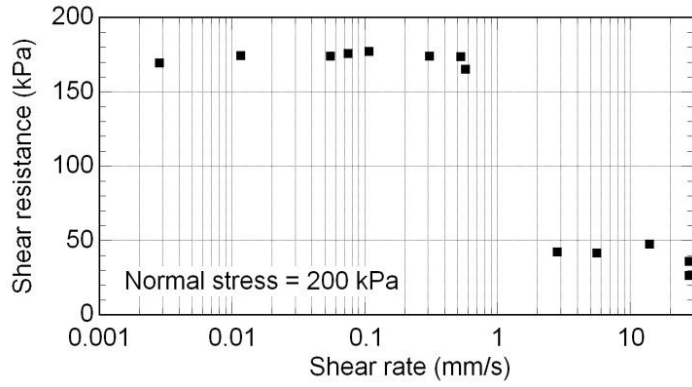


Fig. 10

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771 Fig. 11

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