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Posted Date: 24 April 2024

doi: 10.20944/preprints202404.1577.v1

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Motion Simulation of Landslides Triggered by Earthquakes Using LS-RAPID Program

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Abstract: Earthquake-triggered landslides are one of the most significant hazards worldwide. These landslides, involving small or large volumes, can develop in debris flow or avalanche with high mobility and long-runout distance. The motion processes of this kind of landslide also quite complicated. This study reproduced the motion processes of two landslides using the LS-RAPID program. The Aso Bridge landslide occurred on April 16, 2016, with rapid and long-runout distance characteristics. The Kataragai landslide occurred on April 9, 2018, travelling with flow-like behavior. Through field investigations and laboratory experiments, features and mechanical parameters of these two landslides were obtained. In the LS-RAPID program, the pre-failure models and motion processes of these two landslides were established and reproduced. Results indicated that sliding speed and travelling morphology at different intervals were considered as two indicators to describe their motion process. However, while sliding speed of these two landslides have significant differences with sliding speeds of actual landslides, the final morphologies after failure were consistent with those observed in the field investigation.

Keywords: earthquake-induced landslides; rapid and long-runout; flow-like; computer simulation; motion process

Introduction

Earthquake-triggered landslides, characterized by high velocity and long-runout distance, are commonly one of the most serious hazards globally [1,2]. This type of landslide not only causes catastrophic damage to nearby infrastructures and even famous scenic areas but also results in a larger number of victims and more significant economic losses [3–11].

The 2016 Kumamoto earthquakes consisted of two foreshocks and one mainshock that occurred consecutively at 21:26 on April 14, 00:03 on April 15, and 01:25 on April 16, 2016 (Japan standard time, JST). These earthquakes caused both direct and indirect impacts, resulting in 273 deaths and 2809 injuries, as well as complete or partial destruction of a significant number of houses, public buildings, roads, and bridges [12]. Xu [13] reported that the mainshock was the cause of all the coseismic landslides, including the Aso Bridge landslide. This landslide, classified as a debris avalanche [14], was the largest of the landslides with rapid and long-runout characteristics [15].

The 2018 Western Shimane earthquake, with a magnitude of M6.1 (JMA), struck Oda City, Shimane Prefecture, Japan, at 01:32 on April 9, 2018 (JST). This event caused 9 injuries and fully or partially destroyed 74 houses [16]. Although this earthquake only triggered two landslides, the Kataragai and Shizumi landslides [17], the Kataragai landslide was notable for its flow-like characteristics and sliding distance of 250 m [18].

The Landslide computer simulation (LS-RAPID) program has been extensively developed and applied to simulate the initiation and motion processes of rapid and long-runout landslides triggered by earthquakes, rainfall, and/or their interactive effects [18–31]. These studies emphasized the crucial and fundamental role of determining soil parameters. Some soil parameters, such as initial apparent friction, accumulation possibility excess pore pressure, lateral earth pressure coefficient, and unit weight of sliding soil, depend on the results of field investigations. Other parameters, including effective friction coefficient of sliding soil, shear resistance of sliding soil at steady state, and cohesion inside sliding soil, are determined by ring shear tests.

The purpose of this study is to examine the motion processes of the Aso Bridge and Kataragai landslides using the LS-RAPID program. Firstly, field investigations were carried out to obtain some soil physical parameters and characteristics of the Kataragai landslides. Secondly, based on previous results from field investigations and ring shear tests of the Aso Bridge landslide [26,32,33]. the soil physical parameters and characteristics were described. Thirdly, other parameters were determined based on previous results of ring shear tests and geological condition [22,26]. Finally, the pre-failure morphologies of these landslides were established using DEM data downloaded from the Geospatial Information Authority of Japan (https://fgd.gsi.go.jp/download/menu.php), and all the soil parameters were applied to this program. During the simulation process, the sliding velocity and motion features of these two landslides were recorded at different time intervals. The validity of these parameters applied to the program could be confirmed by comparing post-failure features of simulations with those observed in field investigations.

Geological Setting

The 2016 Kumamoto earthquakes occurred beneath Kumamoto city, Kumamoto prefecture, Kyushu Island, Southern Japan (Figure 1), which contains the Aso caldera, one of the largest volcanic calderas in the world [34]. This caldera was formed by four instances of volcanic activity, and four pyroclastic flow deposits are known as Aso-1 (270 ka), Aso-2 (140 ka), Aso-3 (120 ka) and Aso-4 (90 ka) in ascending order [34,35]. The mainshock triggered all the coseismic landslides that killed at least 10 people out of the 69 confirmed deaths associated with the earthquake [36]. The Aso Bridge landslide occurred on the steep western wall of the Aso caldera, where the pyroclastic soil is prone to failure during seismic shaking and rainfall [33,37]. After this landslide, one person went missing immediately [14].

Figure 2 depicts the lithologies of Minamiaso village and Aso city, based on a 1:200,000 seamless geological map published by the Geological Survey of Japan (AIST). The post-caldera central cones are covered by Holocene mafic volcanic rocks, Late Pleistocene felsic volcanic rocks and mafic volcanic rocks, while the western wall of the Aso caldera is composed of Middle Pleistocene mafic volcanic rocks and pyroclastic flow volcanic rocks. Additionally, the Aso Bridge landslide area comprises Middle Pleistocene mafic volcanic rocks and felsic volcanic rocks. Song [33] reported that the stratigraphic structure of the landslide consists of five layers from top to bottom: (1) medium brown cohesive soil with fewer lapilli and blocks, (2) black cohesive soil, (3) medium brown lapilli and blocks, (4) black cohesive soil, (5) medium brown cohesive soil with fewer lapilli and blocks. These cohesive soils are products of the Aso-3 and Aso-4 formations [26] and have a porous structure that can absorb and infiltrate water into the lower layer, weakening the strength properties and contributing to susceptibility to failure due to rainfall and seismic shaking.

The 2018 Shimane earthquake occurred in Oda city, Shimane prefecture, located at the Yamain area on the southeast coast of the Sea of Japan in Western Honshu (see Figure 3). Figure 4 delineates the lithology of this area based on a 1:200,000 seamless geological map, published by the Geological Survey of Japan (AIST). It can be seen from this figure that the lithology of the landslide-affected area was successively covered by marine and non-marine sediments of the Early to Middle Miocene and Middle to Late Miocene. Besides, its neighboring areas consist of marine and non-marine sediments of Pleistocene, mafic plutonic rocks and mafic volcanic rocks of Miocene, and granite of Middle Eocene. However, outcropped rocks were not observed in the source area, probably because the area and its surroundings were once covered by clay refilled with medium sand and gravel some years ago.

Preceding Rainfall

Rainfall preceding an earthquake is commonly considered to be a controlling factor in the study of the failure mechanism of earthquake-induced landslides. This is because after rainfall infiltrates the soil, it increases water content and reduces the shear resistance of soil, even causing soil failure.

 doi:10.20944/preprints202404.1577.v1

Soil affected by high water content can travel longer distances than that of landslides without rainfall [38]. Figure 5 shows the daily and cumulative precipitations of Oda city from March 10, 2018, to April 11, 2018, and of Minamiaso village from March 14, 2016, to April 18, 2016. From the figure, prior to the earthquake, the 3-day, 10-day, and 30-day cumulative precipitation of Oda city and Minamiaso village reached 19.5 mm, 33 mm, and 150.5 mm, and 4 mm, 63 mm, and 100 mm, respectively. Although the cumulative precipitations of the Kataragai and the Aso Bridge landslides do not significantly differ, the hydrological condition of the Kataragai landslide is much worse than that of the Aso Bridge landslide. This is because the annual precipitation in Oda city, based on precipitation data from 2008 to 2018, was approximately 1752 mm, which led to four ponds having been full of water before the failure of the Kataragai landslide. As a result, the water content of sliding mass was at a totally saturated steady state, and the shear strength of the sliding mass decreased. From the observation of field investigation. three ponds on the crown were unchanged, and one pond in the source area was destroyed. Therefore, after the slope initiated, the destroyed pond caused the sliding mass to travel the distance of 250 m.

Materials and Methods

Field Investigation

Field investigations, including observations of geological conditions, determination of the landslide boundary, measurement of the landslide's longitudinal profile, portable dynamic cone penetration test, hardness test, and sampling, were carried out. The locations of portable dynamic cone penetration tests (No.1~No.13), hardness tests (No.14, 15), and sampling (No.16) were marked in Figure 10. The longitudinal profile of the post-failure slope was measured using a laser range finder, a ranging rod, an inclinometer, and a GPS tracker. The portable dynamic cone penetration test can be used to mark the locations of the water table and determine the sliding surface [33,37]. This test was performed in two steps: a 5-kg hammer is dropped freely from a height of 0.05 m into the soil, and the counts of the dropping hammer to each 0.01-m depth of the portable dynamic cone tip are recorded simultaneously [37,39]. The correlation between depth and compressive strength was explained using the hardness test [40]. The formulation used for this test is expressed as follows:

$$q_{\mathrm{u}} = 1000x/0.7952(40 - x) \tag{1}$$

where, q_{u} is the uniaxial compressive strength (kPa), and x is the reading of the hardness tester inserted into the soil (mm).

Laboratory Experiments

Laboratory experiments were performed according to second edition of the standards of the Japanese Geotechnical Society [41]. According to this book, the grain size distribution of samples taken from the field was conducted. This book also introduces formulations of uniformity coefficient (C_{u}) and curvature coefficient (C_{v}), and permeability coefficient (k) of soil and their equations are expressed as follows:

$$C_u = \frac{D_{60}}{D_{10}} \tag{2}$$

$$C_v = \frac{D_{30}^2}{D_{10} \times D_{60}} \tag{3}$$

$$k = 100 D_{10}^2 \times \frac{1}{100} \tag{4}$$

Where, D_{10}, D_{30}, D_{60} represents grain size when the amounts of grain content are greater than 10 %, 30%, 60 % respectively.

LS-RAPID Program

 doi:10.20944/preprints202404.1577.v1

LS-RAPID program is commonly used to simulate the initiation and motion of landslides caused by earthquakes, rainfalls, and/or their interacting effects [24]. The principle of this program lies in the derivation of motion equation from the equilibrium of forces acting on a soil column, and the continuum equation from fluid dynamics [19]. Before setting up this program, critical mechanical parameters consisting of apparent friction angle (φ_a), shear resistance at the steady state (τ_ss), and accumulation possibility excess pore pressure ratio (B_ss) need to be determined.

The apparent friction angle (φ_a) is introduced to describe a relationship between the friction angle of the sliding surface and the built-up pore water pressure during shearing [42]. This angle is related to the average coefficient of friction, which is the ratio of between the total height (H) and the total travel distance (L) [43].

According to Varnes [44], the sliding zone is composed of two layers: the debris layer (upper layer) and the sliding layer (lower layer). During motion, the thickness of the debris layer gradually decreases until the sliding mass reaches a distance where the shear resistance (τ_ss) remains stable, which can be directly measured using the ring shear apparatus [45].

The B_ss parameter is significantly influenced by the groundwater situation and the drainage condition of the sliding zone. To quantify this parameter, three types (A-type, B-type and C-type) were defined by simplifications of the failure behaviors of the sliding zone and the generation and dissipation of pore water pressure during shearing. In the A-type, the sliding soil remains dry and cannot build up the pore water pressure during shear. In this type, the shear resistance and apparent friction angle are equivalent to the residual strength and residual friction angle at a steady state, respectively. In the B-type and C-type, the sliding soil is saturated and can build up pore water pressure. The significant difference between these two types is the speed of pore water pressure generation and dissipation. The former has a generating speed significantly greater than the dissipating speed, which can form an undrained state, while the latter has a generating speed lower than the dissipating speed, which is referred to as a partially drained state. Based on the three types, the range of this parameter is from 0 to 0.1 if the unsaturated sliding mass travels on a fully dry surface under the A-type. If the saturated sliding mass flows on an impermeable surface under the C-type, its range is 0.9 to1.0. If the saturated sliding mass moves on a permeable surface or the unsaturated sliding moves on a fully saturated surface under the B-type, its range is 0.1 to 0.9.

Results and Discussion

Controlling Factors

Figures 6 and 7 show general aerial view (post-failure) and observations of geological conditions of Kataragai landslide, respectively. The pond located in the source area was destroyed by the sliding mass, exposing sandy gravel soil mixed with water (Figure 6a). Besides, traces of water seepage and flow were observed near this destroyed pond (Figure 6b–d). During the sampling process, a phenomenon was observed in which a spring gushed out of the sliding layer. These observations confirmed that four ponds being full of water contributed to the sliding mass being in a long-term state of saturation before the earthquake. Furthermore, Li [17] conducted a self-potential test on the crown which verified the saturated state of the sliding soil after the slope failure, further confirming the poor hydrological condition of the soil.

Figures 8 and 9 show general aerial view and contour map, as well as the soil layer and the gorge of the Kurokawa River, respectively. From the Figure 8, the Aso Bridge landslide destroyed the National Route 57, the JR Hohi Railway, and the underground water supply channel, and collapsed the 200-m long Aso-Ohashi Bridge that spans an 80-m-deep gorge of the Kurokawa River near the Aso caldera [32], Water seeping out of the failure surface was also found within one day after the failure of slope. Additionally, the sliding mass of the Aso Bridge landslide had a volume of approximately 183,0000 m^3 [33], with most of it sliding into the Kurokawa River, which prevented the formation of a landslide dam [46]. Goda [47] also confirmed that a small mass of debris from the source area had travelled to the opposite riverbank of Kurokawa River prior to the collapse of the Aso Bridge. This suggests that the driving force and/or weight of the sliding mass directly contributed to the collapse of Aso Bridge and could also be attributed to the high travelling velocity of the sliding

 doi:10.20944/preprints202404.1577.v1

mass, resulting in built-up pore water pressure during shearing that could not be immediately expelled. The bedrock of the affected landslide area was primarily composed of lava and volcanic rocks from eruptions that occurred approximately 90,000 years ago [33]. These rocks have formed cohesive soil with porous voids due to weathering, which can absorb and infiltrate water into the lower layer of the soil, reducing the resistance strength of the soil.

Figure 11 presents the longitudinal pre-failure and post-failure profiles of the Kataragai landslide. As shown in the figure, the landslide has an apparent friction angle of 7°, indicating high mobility. Additionally, the landslide travelled approximately 250 m, and its sliding direction was altered due to changes in topography during the motion process. According to the field investigation of the Aso Bridge landslide [33], the slope angle was measured to be 38.6°, and the apparent friction angle was estimated to be approximately 23°, as well as the slope travelled distance of approximately 705 m. However, the Aso Bridge landslide did not change the sliding direction during the motion process.

Figures 12 and 13 show the relationship between N_{d}-value and depth. From Figure 9, it can be seen that except for No.3, all the values of N_{d} (Nos. 1-10) at depths of 0-2 m were less than 10, indicating that the sliding mass is very loose and has relatively low shear resistance [49,50]. The N_{d} value of No. 3 at depth of 1 m was approximately 16, which might be related to the little sliding mass deposited there due to the change of sliding direction. Notably, only No.9 showed the location of the sliding surface which was located at 1.8 m. From the Figure 10, results of No.11 and No.13 showed the locations of sliding surface near depths of approximately 1.65 m and 1.6 m, respectively, where values of N_{d} were 21 and 16, respectively. Besides, N_{d} values increased dramatically at depths greater than 1.5 m. Therefore, the location of sliding surface of the kataragai landslide was thought to be at a depth of 1.6 m to 1.8 m. Song [38] proposed a relationship between the N_{d} value and depth of the landslide soil at various sites, which presented that the sliding surface of the right flank of the Aso Bridge landslide was located at a depth of 6.5 m, three times deeper than that of the Kararagai landslide. This indicates that the Aso Bridge landslide has more potential energy to transform to kinetic energy than that of the Kataragai landslide. The N_{d} value of the sliding surface of the Aso Bridge landslide was estimated to be approximately 20 that is higher than that of the Kataragai landslide. Therefore, the Aso Bridge landslide has a better geological condition and higher shear resistance.

Figure 14 shows the diagram of uniaxial compressive strength versus depth. The uniaxial compressive strength values at No.14 were relatively small (less than 500 kPa) at different depths, while the uniaxial compressive strength values at depths of 1.65-1.85 m in the location of No.15 reached the lowest ranges. This may be because the location of No.14 is closer to the pond, which probably resulted in an increase of water content of neighboring soil.

Figure 15 delineates the grain size distributions of four samples taken from No.16 at different depths. This figure indicates that the sample from the sliding layer contained grain sizes of 15% (>2 mm), 76.5% (0.075~2 mm), and 8.5% (< 0.075 mm), which is classified as as gravel sand with fine (SG-F) according to JGS [41]. For the Aso Bridge landslide, the Aso-3 and Aso-4 of samples from sliding layer both contained grain size of approximately 96% (0.075~2 mm) and 4% (<0.075 mm) [26], which were classified as sand (S).

Table 1 presents the uniformity coefficients, curvature coefficients, and permeability coefficients of four soil samples of the Kataragai landslide and Aso-3 and Aso-4 samples of the Aso Bridge landslide. According to JGS [41], the uniformity coefficient greater than 10 and the curvature coefficient between 1 and 3 indicate good gradation sand or gravel. From Table 1, for the Kataragai landslide, C_{u} was 13.4 and C_{v} was 0.57 at a depth of 1.6 m, indicating that the grain size distribution of sliding soil at this location was continuous gradation sand. Besides, values of C_{u} decreased with increasing depth, suggesting that the grain size become more uniform. The permeability coefficients of the four samples also decreased with increasing depth, and the permeability coefficient of the sliding soil of sliding surface is 1.21×10^{-2} m/s, indicating that the sliding layer was highly permeable. The uniformity coefficients and curvature coefficients of Aso-3 and Aso-4 samples of the Aso Bridge landslide were significantly different. Specifically, the Aso-3 sample exhibited

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characteristics of good gradation sand. Additionally, permeability coefficients of Aso-3 and Aso-4 samples were both 6.25×10^{-4} m/s, indicating that the sliding layer was lower permeable.

Table 2 summarizes the different features of the Aso Bridge and Kataragai landslides obtained according to field investigations and laboratory experiments. The elevation of the Aso Bridge landslide was ten times that of the Kataragai landslide, making it a large-scale landslide compared to the small-scale Kataragai landslide. The sliding-surface liquefaction mechanism of the Aso Bridge landslide also suggests that it could travel longer distances than the Kataragai landslide. The unchanged sliding direction of the Aso Bridge landslide during its traveling process was one reason for its more rapid motion speed. However, the Kataragai landslide, with a very lower apparent friction angle value of about 7°, exhibited high mobility, enabling it to travel 250 m.

Parameters Setting

The LS-RAPID program (version 1.10) was utilized to simulate the motion processes of two earthquake-triggered landslides, namely the Aso Bridge and Kataragai landslides. The topographic data, including slope elevations and sliding surface elevations, were essential for establishing the pre-failure models of these landslides. DEM data for both landslides were downloaded from the Geospatial Information Authority of Japan (https://fgd.gsi.go.jp/download/menu.php). Various data, such as point data, mesh data, axial and mesh setting of the calculation area, and the possible maximum numbers of control points, were inputted into the program.

To establish the pre-failure morphologies the Aso Bridge and Kataragai landslides, DEMs with different resolutions were downloaded. A 10-meter-grid resolution DEM was utilized for the large-scale Aso Bridge landslide, with the following axial settings: x-direction Min. = 0.000 m, Max. = 1010 m; y-direction Min. = 0.000 m, Max. = 570 m. This resulted in a total of 102 (x-direction) × 58 (y-direction) grids and the possible maximum of 10000 control points.Conversely, a 5-meter-grid resolution DEM was downloaded for the small-scale Kataragai landslide, with axial settings: Min. = 4481 m, Max. = 4964 m; y-direction Min. = 3140 m, Max. = 3599 m. This also resulted in 162 (x-direction) × 154 (y-direction) grids and the possible maximum of 10000 control points. The pre-failure models for both landslides were established and shown in Figure 16, with the original sliding mass area represented by a white solid circle in the models.

The mechanical parameters entered this program were obtained from the field investigations and laboratory experiments.

For the steady-state shear resistance ($\tau_{\rm ss}$), Sassa [22] summarized that its scale was 20 to 50 kPa by a series of ring shear results. The sliding soil of the Aso Bridge landslide was mainly dominated by an active volcano and weathered cohesive soil and Dang [26] also measured the shear resistance of 45 kPa at the steady state of sliding soil using the ring shear apparatus. Therefore, its steady-state shear resistance of sliding soil was estimated to be 50 kPa. For the Kataragai landslide, the sliding soil was characterized by a low shear resistance due to $N_{\rm d} < 10$. The sliding zone soil had been saturated for a long time due to the neighboring ponds that contributed to the slope destabilization. Therefore, its shear resistance of $\tau_{\rm ss} = 20$ kPa was assigned.

The lateral pressure ratio K is an indicator of the "softness" or "potential for lateral spreading" of the sliding mass, as proposed by Sassa [48]. This value is based on lateral pressure ratio at rest proposed by Jaky [49Song]. Sassa [19] derived it using the apparent friction angle within the soil mass to obtain this value, as expressed in Equation (5):

$$K = 1 - \sin\varphi \tag{5}$$

where φ is the apparent friction angle and K is the lateral pressure ratio.

The field survey measured the apparent friction angle of 23° and 7° for the Aso Bridge and Kataragai landslides, respectively. These values were used to obtain the respective apparent friction coefficients of 0.61 and 0.88 using Equation (2). Typically, the lateral pressure ratio can be calculated based on the apparent friction coefficient, but due to the saturated state of the sliding mass in both landslides, the lateral pressure ration range of 0.65~0.85 was assumed (Sassa et al., 2004). Thus, the

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Kataragai and Aso Bridge landslides were assessed to have lateral pressure ratios of 0.7 and 0.85, respectively.

The parameter B_{ss} is influenced by groundwater situation and drainage condition of the sliding zone. For the Aso Bridge landslide, a spring seeping from the sliding surface was observed [26], indicating saturation before the slope failure. The sliding zone soil was predominately composed of active volcanic soil and weathered volcanic cohesive soil, which had low permeability and led to a high pore pressure generation rate (B_{ss}) of 0.8 for the Aso Bridge landslide. Initially, the estimated value of the parameter B_{ss} for the Kataragai landslide was 0.2, which was because the saturated sliding mass flowed on a permeable surface composed of gravel with fine sand. However, as the saturated sliding mass traveled a certain distance, the permeable surface transitioned into an impermeable one that was covered by the pitch. As a result, the parameter B_{ss} increased to 1.0 to account for the limited drainage and high pore pressure generation in the sliding process.

The laboratory experiments determined the natural total unit weights of the sliding soil to be 20 kN/m^3 and 18 kN/m^3 for the Aso Bridge and Kataragai landslides, respectively.

Table 3 provides the recommended range of the friction angle for the sliding zone, which varied from 25° to 35°. The friction angle of the Aso Bridge landslide was 25° due to sliding-surface liquefaction, while the Kataragai landslide had a friction angle of 35° due to collapse structure liquefaction. Consequently, the friction coefficients for the sliding soil of the Aso Bridge and Kataragai landslides were 0.577 and 0.700, respectively. In this study, the initial apparent friction angle was assumed to be equal to the measured apparent friction angle in the field. The Aso Bridge and Kataragai landslides had measured apparent friction angles of 23° and 7°, respectively, corresponding to apparent friction coefficients of 0.4 and 0.1, respectively. Based on the soil composition of the Aso Bridge and the Kataragai landslides, their cohesions of soil inside the landslide mass were assumed to be 0 Kpa. Table 4 summarizes all the mechanical parameters of the Aso Bridge and Kataragai landslides in the LS-RAPID program.

Figure 15 illustrates the establishments of the 2D and 3D pre-failure morphologies of these two landslides. Models showed that the total volume of sliding mass (the red solid circle) were 588,681 m^3 (Figure 16a) and 5,726 m^3, respectively. Mechanical parameters were utilized to reproduce the motion processes of the Aso Bridge and Kataragai landslides. Subsequently, the simulation results, such as the velocities in the x- and y-directions and the motion characteristics of the sliding mass, were recorded at different time intervals. The resultant velocity, which combines with the velocities in x- and y-directions, is defined as the comprehensive reflection of the whole landslide's velocity.

Figure 17 depicts various velocities (Umax, Vmax, and Resultant) of the Aso Bridge landslide during its motion. From the figure, the entire motion process lasted for 47.8 s. As the motion time was 24.8 s, the resultant velocity peaked at 297.1 m/s, indicating the rapid characteristics of this landslide. After the motion time reached 47.8 s, most of the sliding mass moved downslope and rushed into the Kurokawa River, with only a little mass remaining on the toe of the slope. The motion process went through two phases: the increasing and decreasing phases.

Figure 18 shows morphologies of 11.8, 24.8, 36.8, and 47.8 s. At the motion time of 11.8 s, the velocities in the x- and y-directions rose to $U\text{max} = 22.4$ m/s and $V\text{max} = 15.0$ m/s, respectively, when some of the sliding mass in the source area (the red solid circle) moved down and buried the underground water supply channel (Figure 18a). When the resultant velocities increased to 297.1 m/s, the area affected by the sliding mass had sharply enlarged and destroyed the JR Hohi Railway and National Road 57 (Figure 18b). As the sliding mass continued to slide, the velocities from different directions began to decrease due to the energy consumption. The affected area continued to expand as the cluster of the sliding mass downslope gradually rushed into the deep gorge of the Kukawa River, causing the Aso bridge to collapse (Figure 18c). At $t = 47.8$ s, most of the sliding mass deposited downslope, which largely matches the geographic deposition of the actual landslide (Figure 8b).

Figure 19 presents various velocities (Umax, Vmax, and Resultant) of the Kataragai landslide. This event had a total motion time of approximately 48.9 s. When the motion time reached 32.6 s, the maximum resultant velocity was only 43.6 m/s, which was much lower than that of the Aso Bridge

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landslide. This was due to a lower driving force, smaller volume of sliding mass, and gentle slope, despite the geological and hydrological conditions being much worse than those of the Aso Bridge landslide. The motion processes of this landslide underwent three stages in this simulation: the increasing, intermediate, and decreasing stages. This result corresponds to the change in sliding direction, as shown in Figure 6. In Figure 18, in the increasing stage of the motion process, the resultant velocity ranged from 0 to 33.5 m/s, followed by a range of 33.5 to 22.6 m/s, and then 22.6 to 43.6 m/s in the intermediate stage. Finally, the velocity reduced to 0.5 m/s in the decreasing stage. Thus, four typical motion times were represented to depict the motion features of this landslide (Figure 6).

As shown in Figure 20, after the slope started to initiate, the sliding mass in the original area (the white solid circle) flowed down and rushed out of this area. When the motion time reached 23.6 s, the affected area (the red solid circle) expanded, and the sliding mass flowed toward the toe of the slope. When the velocities increased to their maximum values, the sliding mass flowed directly into the farmland and wasteland in front of the toe of the slope. When the velocities were reduced to their minimum values, the motion slowly stopped, and the sliding mass deposited somewhere. Due to the low driving force and the gentle slope angle, most of the sliding mass left the source area and stopped at the deposition area. This was why the N_{d}-values of the source and deposition areas were very small. Throughout the entire motion process, no significant damage was noted in the neighboring area due to the low velocity, small sliding mass volume, and changing sliding direction of this landslide. The motion features along the entire paths of this landslide in the model were also like the results of the field investigation (Figure 6).

Conclusion

This study investigated field features of the Kataragai landslide, triggered by the 2018 Western Shimane earthquake and compared it with field investigations of the Aso Bridge landslide, induced by the mainshock of the 2016 Kumamoto earthquakes. Mechanical parameters were obtained from field investigation and laboratory experiments and were applied to the LS-RAPID program to simulate motion processes of these two landslides. Conclusions drawn from this study are as follows:

1) Controlling factors, including apparent friction angle, slope angle, hydrological condition, change in sliding direction, and liquefaction mechanism, contributed to significant differences in failure destructiveness, sliding speed, motion features, and sliding distance between these two landslides.
2) The simulation result demonstrated that motion features of the entire traveling path of these two landslides were consistent with those that of the field investigation.

Funding: This paper is financially supported by the Fundamental Research Grant (2017-2019) of Shimane University on “development of prediction and mitigation technologies on natural disasters in subduction zone using San-in region as a research field” and the fund “initiation and motion mechanisms of long run-out landslides due to the rainfall and earthquake in the falling pyroclastic deposit slope area” (JSPS-B-19H01980, Principal Investigator: Fawu Wang).

Data Available Statement: The dataset associated with the slope elevations and sliding surface elevations of the Kataragai landslide were freely downloaded from the Geospatial Information Authority of Japan (https://fgd.gsi.go.jp/download/menu.php).

Acknowledges: The authors sincerely appreciate Fawu Wang (Tongji University, Shanghai, China), Tetsuya Sakai (Shimane University, Matsue, Japan), Ran Li and Shuai Zhang (Institute of Geomechanics, Chinese Academy of Geological Sciences, Beijing, China), and Zili Dai (Shanghai University, Shanghai, China) for their kind assistant in the field work and during COVID-2019. The author also gratefully acknowledged the financial support from China Scholarship Council (CSC) to study in Shimane University, Japan.

Conflicts of Interest: The author declares no conflicts of interests.

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