

**DEVELOPMENT OF HAZARD ASSESSMENT
TECHNOLOGY OF THE PRECURSOR STAGE
OF LANDSLIDES**

Lam Huu Quang

2018

Development of Hazard Assessment Technology of the Precursor Stage of Landslides

**(前兆段階にある地すべりの災害危険度
評価技術の開発)**

by

Lam Huu Quang

A dissertation

**Submitted in partial fulfillment of the requirements for the
Degree of Doctor of Engineering**

**Department of Civil and Earth Resources Engineering
Kyoto University, Japan**

November 2018

Acknowledgements

Firstly, I would like to express my sincere gratitude to my respected professors, Professor Emeritus Kyoji SASSA, Executive Director of the ICL, and Professor Kaoru TAKARA, who accepted me to become a doctoral student in his Laboratory. They have continuously guided, supported and advised me in all throughout the research phase and writing of this thesis. I cannot imagine having a better advisor and mentor for my Ph.D study.

Besides my advisor, I would like to thank the rest of my thesis committee: Dr. Dang Quang Khang, Dr. Hirotaka Ochiai, Dr. Shinro Abe, Dr. Shiho Asano, Mr. Doan Huy Loi, Mrs. Tran Thuy Anh, Mr. Do Ngoc Ha, Mr. Luu Tuan and Mr. Pham Van Tien for their insightful comments and encouragement, but also for the hard question which incited me to widen my research from various perspectives. Without their precious support it would not be possible to conduct this research.

My sincere thanks also go to Assoc. Prof. Nguyen Xuan Khang, Director of the Institute of Transport Science and Technology (ITST) who invited me to join to a Japan-Vietnam research project as a leader of testing group in Vietnam side and giving me a chance to study this doctoral course at Kyoto University. I would like to give my thanks to colleagues in ITST: Dr. Dinh Van Tien, Dr. Bui Ngoc Hung, Mr. Huynh Dang Vinh, Mr. Huynh Thanh Binh, Mr. Nguyen Kim Thanh, Dr. Le Hong Luong, Mr. Tran Dang Ninh and Ms. Pham Thi Chien for their useful suggestions during this research.

A special thanks to my family. Words cannot express how grateful I am to my mother-in law, father-in-law, my mother, and father for all of the sacrifices that you've made on my behalf. Your prayers for me were what sustained me thus far. I would also like to thank all of my friends who supported me in writing, and encouraged me to strive towards my goal. Last, but not least, I am really grateful to my wife Le Ngoc Linh, my son Lam Huu Binh and my daughter Lam Ngoc Minh for their love, support and encouragement that enabled me to complete my course.

Abstract

Landslide is considered as one of the most widespread hazards on Earth that causes thousands of worldwide deaths and injuries estimated at billions of dollars in damage per year (Kjekstad, et al. 2009, Baum, et al. 2014). Landslide disasters have appeared increasingly around the world triggered by both rainfalls and earthquakes (Tonje, et al. 2016). The initiation and motion of rapid landslides are complicated processes. The mechanism of rapid landslide is proposed as for a slope at critical stress condition, with an external disturbance such as earthquake or rainfall or combination both factors, the soil at sliding surface would fail.

ICL-1 and ICL-2 were developed as a part of Japan-Croatia and Japan-Vietnam research projects, respectively. Two apparatus were created to simulate the initiation and motion of rainfall earthquake and rapid landslide. ICL-2, the new high-stress undrained ring-shear apparatus with an undrained capacity of 3 MPa, was donated to Vietnam side within the joint project's framework in 2015. The results from two real case-studies (rapid landslide at Ha Long City, Quang Ninh, Vietnam and Kumamoto Earthquake, Japan) demonstrate that the LS-RAPID model predicts a similar hazard area to that observed in the field investigation. In addition, the time of landslide occurrence estimated from the rainfall record and the LS-RAPID simulation is close to the time of occurrence reported by local inhabitants.

Following two real case-studies of rapid landslides, the combination of ring shear apparatus and LS-RAPID was applied to a precursor stage of a landslide at the Haivan Station in Vietnam. The Haivan Station is an important station on the North-South railway line in central Vietnam. Field investigation has identified a precursor stage of a landslide that would threaten this railway. Therefore, a landslide hazard assessment for Haivan Station was urgently needed to protect passenger safety and the national railway. Conducted investigations include air-photo interpretation, drilling, ground water and inclinometer monitoring, laboratory testing, and landslide simulation.

By literature review, a series of undrained ring-shear devices with load capacities of less than 1 MPa has been developed since 2004 and practised on various landslides (Sassa, et al. 2004a, 2004b, 2010, 2014a, Boldini, et al. 2009, Arbanas, et al. 2014,

Yin, et al. 2016, Dang, et al. 2016, Loi, et al. 2017). All samples, except the study for a hypothetical submarine landslide in 2012 (Sassa, et al. 2012), were taken from the landslide ground surface. The combination of landslide ring shear simulator and integrated landslide simulation model is considered as a new tool for landslide assessment. However, this technology has not been applied to the precursor stage of any landslides yet.

In this research, the new landslide assessment with utilizing ICL-2 was implemented for precursory phenomena to predict and estimate landslide disaster occurrence. The case study was performed on a precursor stage landslide threatening Haivan railway station in Vietnam. It is a difficult task to present a landslide hazard assessment of a slope which could endanger a very important facility such as a railway station or a cultural heritage. This is the first research using drill-core samples taken from a precursor stage of a developing sliding zone to measure landslide dynamic parameters, conduct LS-RAPID simulation and assess the landslide hazard for an essential infrastructure (Lam, et al. 2017).

The landslide dynamic parameters obtained from the ring-shear test of the 50-m deep sample were used in an integrated numerical simulation model LS-RAPID. The simulation result gave the critical pore-pressure ratio for landslide occurrence, and landslide's likely maximum speed, total volume, and depth of landslide debris that could cover the railway. These estimates serve to raise awareness of the vulnerability of the Vietnam national railway sector to landslide impact.

Table of Contents

Acknowledgements	i
Abstract	iii
Table of Contents.....	v
List of Figures	ix
List of Tables	xiii
Abbreviations	xiv
Chapter 1 Introduction	1
1.2 Background.....	5
1.3 Objectives of this study	7
1.4 Thesis Outline.....	7
Chapter 2 Study of the Mechanism of Rainfall-induced Rapid Landslides	9
2.1 Introduction	10
2.1.1 The Mechanism of Rain-induced Rapid Landslides	10
2.1.2 Study Area-The 28 July 2015 Rapid Landslide at Ha Long City, Quang Ninh, Vietnam	14
2.2 Methodology.....	16
2.2.1 SLIDE Model	16
2.2.2 UAV method for landslide mapping	18
2.2.3 Ring-shear Simulator for Landslides Triggered by Rainfall	19
2.2.4 Integrated Simulation Model LS-RAPID for the Initiation and Motion of Landslides Triggered by Rainfall	20
2.3 Sampling, Testing and Results	21
2.3.1 Sampling.....	21
2.3.2 Un-drained Monotonic Shear Stress-controlled Test	22
2.3.3 Pore-water Pressure-controlled Test	22

2.4 Application of LS-RAPID and SLIDE Model to the 2015 Ha Long Landslide	23
2.4.1 Triggering Factors	23
2.4.2 Landslide Dynamics Parameters	24
2.4.3 Simulation Result of the Landslide	25
2.5 Summarization.....	27
Chapter 3 Study of the Mechanism of Earthquake-induced Rapid Landslides	29
3.1 Introduction	29
3.1.1 The Mechanism of Earthquake-induced Landslides	29
3.1.2 Study Area-Landslides induced by the 16 April 2016 Kumamoto Earthquake, Japan.....	31
3.2 Methodology.....	36
3.2.1 Ring-shear Simulator with Simulated Earthquake Waveform for Landslides Triggered by Earthquake.....	36
3.2.2 Integrated Simulation LS-RAPID for the Initiation and Motion of Landslides Triggered by Earthquake.....	36
3.3 Sampling, Testing and Results	36
3.3.1 Sampling.....	36
3.3.2 Un-drained Cyclic Loading Test	38
3.3.3 Un-drained Seismic Loading Test.....	39
3.4 Application of the Computer Simulation Model (LS-RAPID) to the Kumamoto Landslides.....	41
3.4.1 Landslide Dynamics Parameters	41
3.4.2 Simulation Result of the Landslide	44
3.5 Summarization.....	46
Chapter 4 Hazard Assessment of the Precursor Stage of Landslides	47
4.1 Introduction	47
4.1.1 The precursor stage of Landslides.....	48

4.1.2 Study Area-A Precursor Stage Landslide Threatening Haivan Railway Station, Vietnam	49
4.2 Monitoring System	55
4.2.1 Borehole Inclinator	55
4.2.2 Rainfall and pore-water pressure measurements	58
4.3 Sampling, Testing and Results	59
4.3.1 Sampling	59
4.3.2 Drain, shear-speed Controlled	60
4.3.3 Undrained, shear-stress Controlled test	61
4.3.4 Pore-water pressure controlled test	62
4.3.5 High-stress, Dynamic-loading, Un-drained Ring-shear Test	63
4.4 Application of the Computer Simulation Model (LS-RAPID) for Hazard Assessment of the Precursor Stage of a Landslide	65
4.4.1 Landslide Dynamics Parameters	65
4.4.2 Simulation the Landslide Motion-the Case of Haivan Landslide	67
4.4.3 Landslide Hazard Assessment for Haivan Station	71
4.5 Summarization	73
Chapter 5 Conclusions	75
References.....	79

List of Figures

Figure 1.1 Global map of reported landslides with fatalities from 2007-2013	1
Figure 1.2 The town of Longarone, before and after the landslide	3
Figure 1.3 Vargas tragedy, Venezuela on 16 December 1999	4
Figure 1.4 The front view of the Leyte landslide on 17 February 2006	5
Figure 2.1 Investigating and sampling in field after the 2015-catastrophic landslide occurred at Cao Thang ward, Ha Long city, Quang Ninh province, Vietnam.....	10
Figure 2.2 Rainfall and groundwater formation and pore-water pressure (Sassa, et al. 2013).....	10
Figure 2.3 Pore-water pressure in the ground water flowing parallel to a slope (Sassa, et al. 2013).....	11
Figure 2.4 Pore-water pressure acting on a potential sliding surface from an underlying layer (Sassa, et al. 2013)	12
Figure 2.5 Landslide-initiation mechanism due to ground-water rise / pore-water pressure rise (Sassa, et al. 2013).....	12
Figure 2.6 Stress acting on the sliding surface within a simple slope (Sassa, et al. 2013).....	13
Figure 2.7 Location map of Ha Long landslide (from Google Earth).....	14
Figure 2.8 Geological map of study area and sampling	15
Figure 2.9 Schematic illustration of water infiltrating an infinite slope (from Liao, et al. 2010b).....	16
Figure 2.10 Pore-water pressure calculation by the SLIDE model from the rainfall record monitored at the Bai Chay weather station for each 1 hour from 0:00 hr on 26 July 2015.....	18
Figure 2.11 UAV image (Modified from Zing) (a) and A-B longitudinal cross section of Ha Long landslide (b)	19
Figure 2.12 Undrained monotonic stress-controlled test on the brown sand sample ($B_d=0.95$, normal stress=95 kPa, $\Delta\tau/s=0.5$ kPa/s). Stress path and time series data (a), Relationship between shear stress and shear displacement (b).....	22

Figure 2.13 Pore pressure-controlled test on the brown sand sample ($B_d=0.98$).....	23
Figure 2.14 Simulation result of Ha Long landslide	26
Figure 2.14 (continued)	27
Figure 3.1 Landslide-initiation mechanism due to effect of earthquake (Sassa et al. 2013).....	30
Figure 3.2 Landslide-initiation mechanism by the combined effect of earthquake & pore-pressure rise (Sassa et al. 2013)	31
Figure 3.3 Monitored peak acceleration of the M7.3 Kumamoto earthquake (from the National Research Institute for Earth Science and Disaster Resilience, NIED) and locations of the epicenter, earthquake record stations, and the two studied landslides (from Google Earth images).....	32
Figure 3.4 Plan and central cross section of the Takanodai landslide area, Minamiaso village, Kumamoto Prefecture (the map was made from UAV images using Agisoft PhotoScan software).	33
Figure 3.5 Plan and section of the Aso-ohashi landslide (the plan is an orthorectified image made from UAV images with Agisoft PhotoScan software)	35
Figure 3.6 Sampling site of Aso-1 and Aso-2 (a, b), grain-size distribution (c) and unit weight (d) of the samples taken from the two landslides.....	37
Figure 3.7 Stress path and time series data of undrained cyclic loading test on Aso-1 sample. $B_d=0.94$, frequency of 0.1 Hz, shear stress step of 30 kPa.	38
Figure 3.8 Three components of the M 7.3 Kumamoto earthquake recorded at KMM005 site (From K-net of National Research Institute for Earth Science and Disaster Prevention)	40
Figure 3.9 Hypothesised shear stress during the M7.3 Kumamoto earthquake acting on the sliding surface of the Aso-ohashi landslide.....	40
Figure 3.10 Stress path and time series data of undrained seismic loading test on Aso-1 sample using the simulated EW component of Kumamoto earthquake waveform at site KMM005 ($B_d=0.95$).....	41
Figure 3.11 Relationship between shear stress and shear displacement of the undrained seismic loading test.....	43
Figure 3.12 Simulation result Takanodai landslide (left side) and Aso-ohashi landslide (right side) due to 16 April 2016 Kumamoto earthquake	45
Figure 3.12 (continued)	46

Figure 4.1 Location, topography, geological structure, an estimated precursor stage of deep-seated landslide border on the back-side slope of the Haivan station, in Vietnam. a) The small step where a head scarp of a precursor landslide may be formed; b) A previous landslide in 2007; c) A previous landslide behind the Haivan station.....	50
Figure 4.1 (continued) d) Geological map of wider area e) Simplified engineering geological map f) Geological cross section A-B.....	51
Figure 4.2 Lithological description of the Haivan station borehole and the sampled sections (red outlines: S1, S2, S3 and S4) from weathered layers of the drilling core	53
Figure 4.3 Location map of monitoring sensors on the Haivan station slope.....	54
Figure 4.4 Inclinator measurements at each depth from 15 Sep. 2015 to 15 Mar. 2016	57
Figure 4.5 The section of BH-1 for pore pressure monitoring and BH2 for inclinometer monitoring	58
Figure 4.6 Hourly precipitation and water level in BH1 is formed by pore pressure acting in the zone of 47.4-51.4 m from 22 April 2015 to 29 June 2016	59
Figure 4.7 Grain-size distribution of Haivan drillcore samples taken from depths of 50 m (S1), 31 m (S2), 21 m (S3) and 6 m (S4)	60
Figure 4.8 Stress paths of drained shear-speed control tests on the three drill-core samples. (A) Sample S1; (B) Sample S2; (C) Sample S3, $B_d=0.96-0.99$	61
Figure 4.9 Undrained shear-stress control tests on Haivan drill-core samples with $\Delta\tau/s=1$ kPa/s. (A) Sample S1, Normal stress=780 kPa, $B_d=0.96$; (B) Sample S2, Normal stress=520 kPa, $B_d=0.95$; (C) Sample S3, Normal stress=350 kPa, $B_d=0.96$	62
Figure 4.10 Pore-pressure control tests on Haivan drill-core samples with $\Delta u/s=1$ kPa/s. (A) Sample S1, Normal stress=780 kPa, Shear stress=360 kPa, $B_d=0.99$; (B) Sample S2, Normal stress=520 kPa, Shear stress=200 kPa, $B_d=0.96$; (C) Sample S3, Normal stress=350 kPa, Shear stress=150 kPa, $B_d=0.97$	63
Figure 4.11 Undrained dynamic loading test on sample S4 (6 m). $B_d=0.98$, initial stresses ($\sigma_0=100$ kPa, $\tau_0=27$ kPa).....	64
Figure 4.12 Shear displacement and shear resistance path of undrained stress control tests on drill-core sample S1 (50 m).....	65

Figure 4.13 Landslide deposits with pore-pressure ratio (r_u) equal to (a) 0.20, (b) 0.30, (c) 0.45 and (d) 0.46	69
Figure 4.14 Landslide development process $\Delta r_u=0.0001/s$. (a) $r_u=0.4501$, $t=1$ s; (b) $r_u=0.4510$, $t=10$ s; (c) $r_u=0.4527$, $t=27$ s; (d) $r_u=0.4550$, $t=50$ s).....	70
Figure 4.14 (continued) (e) pore-water pressure ratio and speed of landslide in relation to time.....	71
Figure 4.15 Hazard assessment of the precursor stage of the potential landslide at Haivan station. (a) A-B cross section of the slope before and after the possibly triggered landslide; (b) Landslide hazard area by the LS-RAPID simulation (within the pink color line).	71
Figure 4.16 Critical condition of landslide initiation as a tool for early warning of landslides	73

List of Tables

Table 1.1. The most catastrophic landslides over the world	2
(source: USGS.gov, Wikipedia)	2
Table 2.1. Test program.....	20
Table 2.2. Basic geotechnical test results of the two samples.....	22
Table 2.3. Parameters used in LS-RAPID simulation.....	24
Table 3.1. Properties of Kumamoto soil samples used in LS-RAPID simulation for Takanodai and Aso-ohashi landslides	42
Table 4.1. Monitoring sensors for Haivan station	56
Table 4.2. Parameters of Haivan landslide used in LS-RAPID simulation	66

Abbreviations

BH	Bore Hole
DSM	Digital Surface Model
DTM	Digital Terrain Model
GNSS	Global Navigation Satellite System
ICL	International Consortium Landslide
ITST	Institute of Transport Science and Technology
K-NET	Kyoshin network
NIED	National Research Institute for Earth Science and Disaster Resilience
SATREPS	Science and Technology Research Partnership for Sustainable Development
SLIDE	Slope-Infiltration-Distributed Equilibrium
UAS	Unmanned Aircraft Systems
UAV	Unmanned Aerial Vehicle
UAVs	Unmanned Aerial Vehicles
WG	Working Group

Chapter 1 Introduction

Landslide is considered as one of the most widespread hazards on Earth that causes thousands of worldwide deaths and injuries estimated at billions of dollars in damage per year (Kjekstad, et al. 2009, Baum, et al. 2014). Landslide disasters have appeared increasingly around the world triggered by both rainfalls and earthquakes (Tonje, et al. 2016). Figure 1.1 displays a Global map of reported landslides with fatalities from 2007-2013, highlighting many “hotspot” landslide areas. The map also shows clear distinctions between landslide reports with few or no fatalities compared to reports with many fatality estimates (Kirschbaum, et al. 2015).

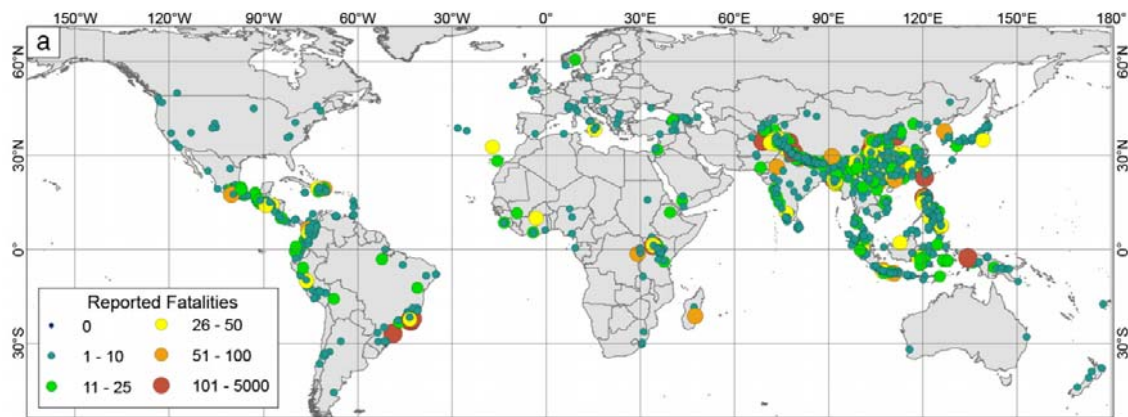


Figure 1.1 Global map of reported landslides with fatalities from 2007-2013
(Kirschbaum, et al. 2015)

Major landslide disasters were listed in the Table 1.1. On December 16, 1920, a $M=8.5$ earthquake occurred near Ganyan Chi, Haiyuan County of the Ningxia Hui Autonomous Region in China (Zhang, 1995). 1920 Haiyuan earthquake is known as the world’s second deadliest disaster of the twentieth century and to have triggered the most disastrous loess earth-flows (Hans-Balder, et al. 2010). Series of catastrophic landslides and loess flows induced by the earthquake over an area of $50,000 \text{ km}^2$, buried villages and killed about 100,000 people.

Table 1.1. The most catastrophic landslides over the world
(source: USGS.gov, Wikipedia)

No	Date	Place	Name	Casualties
1	21 May 1792	Shimabara Peninsula of Nagasaki Prefecture, Japan	1792 Unzen earthquake and tsunami	15,000
2	19 May 1919	Kelud, East Java, Indonesia	Kelut Lahars	5110
3	16 Dec. 1920	Haiyuan County, Ningxia, China	1920 Haiyuan earthquake	>100,000
4	25 Aug. 1933	Diexi, Mao County, Sichuan, China	1933 Diexi earthquake	~3100
5	5 July 1938	Kwansai, Hyogo Prefecture, Japan		~1000
6	13 Dec. 1941	Huaraz, Ancash, Peru	Huaraz debris flow	4,000-6,000
7	10 July 1949	Gharm Oblast, Tajikistan	Khait landslide Yasman valley flowslide	800 ~4,000
8	1953	Wakayama Prefecture, Japan	Arida River landslides	1,046
9	1958	Shizuoka Prefecture, Japan	Kanogawa landslides	1,094
10	10 Jan. 1962	Ranrahirca, Peru	1962 Nevado Huascarán debris avalanche	4,000 - 5,000
11	09 Oct. 1963	Longarone, Italy	Vajont landslide	~ 2,000
12	31 May 1970	Yungay, Peru	1970 Nevado Huascarán debris avalanche	>22,000
13	18 Mar. 1971	Chungar, Peru	Chungar avalanche and tsunami	400-600
14	13 Nov. 1985	Armero, Tolima Department, Colombia	Armero tragedy	23,000
15	16 Dec. 1999	Vargas, Venezuela	Vargas tragedy	30,000
16	17 Jan. 2001	El Salvador	Las Coinas landslide	500-1,700
17	17 Feb. 2006	Southern Leyte, Philippines	2006 Southern Leyte mudslide	1144

No	Date	Place	Name	Casualties
17	17 Feb. 2006	Southern Leyte, Philippines	2006 Southern Leyte mudslide	1144
18	9 Aug. 2009	Xiaolin (or Hsiao-Lin), Kaohsiung County, Taiwan		500-600
19	8 Aug. 2010	Gansu, China	2010 Gansu mudslide	1,287
20	16 June 2013	Kedarnath, Uttarakhand, India	After Uttarakhand Floods	5,700
21	02 May 2014	Ab Barak, Badakhshan, Afghanistan	2014 Badakhshan mudslides	2700
22	13 November 2015	El Cambray Dos, Guatemala Department, Guatemala	2015 Guatemala landslide	220
23	2 April 2017	Mocoa, Colombia	2017 Mocoa landslide	314
24	12 June 2017	Rangamati, Chittagong and Bandarban, Bangladesh	2017 Bangladesh landslides	152

Figure 1.2 shows the landslide of the Vajont Dam occurred in the town of Longarone, Italy in 1963. This rockslide moved into the Vajont reservoir and caused 100 m waves to crash overtop one of biggest dams in the world at the time. Approximately 2,000 people were killed and the economic losses were \$200 million.



Figure 1.2 The town of Longarone, before and after the landslide
(Photograph by the Vajont Survivor's Committee)

The 1999 Vargas tragedy was a disaster that struck the Vargas State of Venezuela on 15 December 1999. 10 days of torrential rains caused flash floods and debris flows that inundated the mountainous regions of the state and other areas

in the northern part of the country. An estimated 190,000 people were evacuated, but likely 30,000 others were killed and the state's infrastructure was completely collapsed (Figure 1.3). Deposits up to 6-m in thickness and totaling about 1.8 million cubic meters of boulder debris were caused by the main channel's avulsion (the left side of Figure 1.3a) and secondary new flood channels were arisen through center of fan (visible at the lower right of Figure 1.3a).



Figure 1.3 Vargas tragedy, Venezuela on 16 December 1999
(Photograph by Lawson Smith, US ACE)

- a. Aerial view of large-scale debris flow deposition resulting in widespread destruction
- b. Apartment in Caraballeda extensively damaged by passage of debris-flow front at least 3.5 m in height, leaving boulders ($> 1\text{m}$) on second floor of structure

A rockslide-debris avalanche occurred on 17 February 2006 in Southern Leyte, Philippines (Lagmay, et al. 2006). This gigantic rapid large-scale landslide claiming 1,144 fatalities was triggered by heavy rainfalls and a small nearby earthquake (Sassa, et al. 2010). The landslide has a platform area of 3,324,400 million square meters, a maximum width of 1.52 kilometers, and a distance of 4.1 kilometers from crown to toe (Lagmay, et al. 2006).

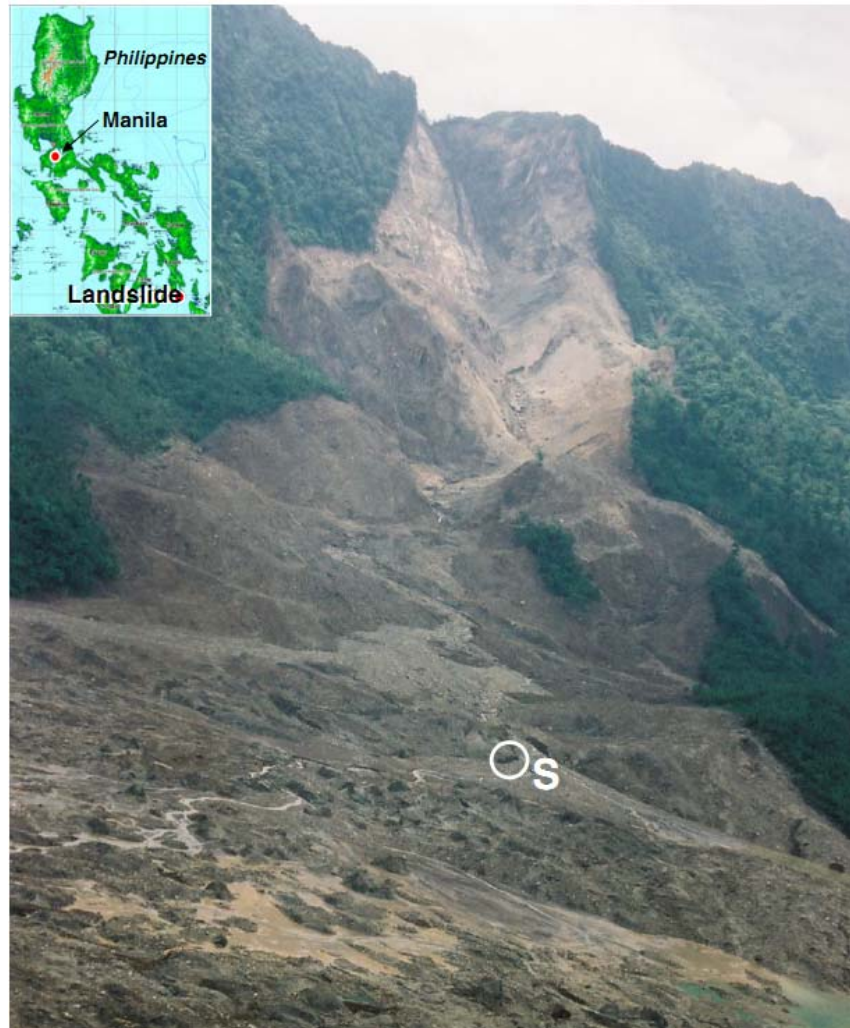


Figure 1.4 The front view of the Leyte landslide on 17 February 2006
(Photograph by K. Sassa from a chartered helicopter)

1.2 Background

In Vietnam, extensive landslides often take place during tropical cyclones. Due to these natural disasters, the annual damage is counted up to around 100 million USD and hence, the ongoing rapid economic development and urbanization of Vietnam can be seriously hampered. It needs to develop methodologies and models to help reducing the vulnerability of the country with respect to landslide disasters.

Relief, recovery and reconstruction are normal solutions that have been implemented in Vietnam and a number of other countries right after a landslide disaster. In fact, these works often take large budgets as well as prolonged time of interruption of socio-economic activities and are not possible to estimate the exact cost. Therefore, developing a landslide disaster management system including

prevention and preparedness components has been attracting increasingly attention and is considered as a manner to save money and time.

Based on this joint research, ICL and Vietnam's Institute of Transport Science and Technology (ITST) successfully applied for a Vietnamese-Japanese joint Science and Technology Research Partnership for Sustainable Development (SATREPS) project "Development of landslide risk assessment technology along transportation arteries in Vietnam". This project consisted of four Working Groups. The first of these (WG1-Integrated research group) was the preparation of integrated guidelines for the application of developed landslide-risk-assessment technology. The second group (WG2-Landslide mapping group) entailed wide-area landslide mapping and identification of landslide-risk areas. The third group (WG3-Testing group) was the development of landslide risk-assessment technology based on soil testing and numerical simulation. The fourth and also final group (WG4-Monitoring group) was risk evaluation and development of early warning based on landslide monitoring (Lam, et al. 2014). The goal of the Project will be obtained through developing of new landslide risk assessment technology, its application to forecast, monitoring and disaster preparedness of landslides in Vietnam.

For landslide risk assessment, it is needed to assess mechanical behavior of the initiation and motion of landslides including hazard area, speed and depth of sliding mass (Sassa, et al. 2014a). Ring shear apparatus were used to simulate the failure of soils, the formation of the sliding surfaces and the steady-state motion of landslides. The new high-stress undrained ring-shear apparatus, ICL-2 has been developed with an undrained capacity of 3 MPa, in the Japan-Vietnam Joint Project of the International Programme on Landslides (Sassa, et al. 2014). An ICL-2 apparatus was donated to Vietnam in 2015. ICL-2 is the first successful 3 MPa undrained ring-shear apparatus. A series of undrained ring-shear devices with load capacities of less than 1 MPa has been developed since 2004. Their test results have been applied to various landslides (Sassa, et al. 2004a, 2004b, 2010, 2014a, Boldini, et al. 2009, Arbanas, et al. 2014, Yin, et al. 2016, Dang, et al. 2016, Loi, et al. 2017). All samples, except the study for a hypothetical submarine landslide in 2012 (Sassa, et al. 2012), were taken from the landslide ground surface.

The 28 July 2015 Rapid Landslide at Ha Long City, Quang Ninh, Vietnam triggered by rainfall and landslides induced by the 16 April 2016 Kumamoto Earthquake were studied to examine the mechanism of rainfall and earthquake-induced rapid landslides. Based on the results of this research and other researches (Sassa, et al.

2004a, 2004b, 2010, 2014a, Dang, et al. 2016, Loi, et al. 2017, Arbanas, et al. 2014, Gradiski, et al. 2013), the combination of landslide ring shear simulator and integrated landslide simulation model is considered as a new tool for landslide assessment. However, this landslide risk assessment technology has not been applied to the precursor stage of landslides. Hence, we applied this technology for precursory phenomena to predict and estimate landslide disaster occurrence. The case study is performed on a precursor stage landslide threatening Haivan railway station, Vietnam. It is a difficult task to present a landslide risk assessment of a slope which could threaten a very important facility such as a railway station or a cultural heritage. This is the first research to use drill-core samples to measure landslide dynamic parameters for a soil taken from a precursor stage of a developing sliding zone, conduct LS-RAPID simulation and assess the landslide risk to essential infrastructure (Lam, et al. 2017). The initiation, motion mechanism and critical condition for landslide initiation are analyzed in Chapter 4.

1.3 Objectives of this study

The main objective of this research is to develop the hazard assessment technology of precursor stage of landslide. There are three specific objectives as follows:

1. Study of the mechanism of rainfall-induced rapid landslides at Ha Long City, Quang Ninh, Vietnam;
2. Study of the mechanism of earthquake-induced rapid landslides in Kumamoto prefecture, Japan; and
3. Development of hazard assessment technology of the precursor stage of landslides.

1.4 Thesis Outline

This thesis consists of five chapters. The content of each chapter is outlined as follows:

Chapter 1 presents the background, the study objectives and thesis outline.

Chapter 2 describes the mechanism of rain-induced rapid landslides. The real study areas of the 28 July 2015 rapid landslide at Ha Long City, Quang Ninh, Vietnam was shown to examine the combination of landslide ring shear simulator and integrated landslide simulation model for the initiation and motion of landslides triggered by rainfall.

Chapter 3 presents the mechanism of earth-induced rapid landslides. The real study areas of landslides induced by the 16 April 2016 Kumamoto Earthquake, Japan was shown to examine the combination of landslide ring shear simulator and integrated landslide simulation model for the initiation and motion of landslides triggered by earthquake.

Chapter 4 provides hazard assessment technology of the precursor stage of a landslide. The case study is performed on a precursor stage landslide threatening Haivan railway station, Vietnam.

Chapter 5 is conclusion derived from this study based on the overall research results

Chapter 2 Study of the Mechanism of Rainfall-induced Rapid Landslides

The four Project Working Groups conducted many research activities related to landslides in Vietnam from 2011 to 2016. As the Vietnam side leader of the WG3-Testing group, the candidate was a member contributed to develop the new high-stress undrained ring-shear apparatus, ICL-2. Then, the WG3 improved LS-RAPID software and provided a new version that allows simulating the formation process of rain-induced rapid landslides based on 10-minute and hourly precipitation data. Moreover, UAV method was applied to build up the Digital Surface Model (DSM) for landslide mapping by the combination of the WG2 – Mapping group and the WG3.

In July 2015, a catastrophic landslide occurred at Cao Thang ward, Ha Long city, Quang Ninh province, Vietnam that claimed 8 lives and destroyed 3 houses. A field investigation was performed by the WG3 to explore the mechanism and development of Ha Long rain-induced rapid landslide. The author and other WG members took soil samples and then tested their physical-mechanical properties in ITST's laboratory in Vietnam while only ring shear test under ICL-1 was conducted by the WG3 members in Japan. Upon Japanese professors' instruction, the candidate analyzed testing results to define inputs of the new LS-RAPID. The main content of this research was published in the Landslide Journal titled as "The 28 July 2015 rapid landslide at Ha Long City, Quang Ninh, Vietnam" and is presented in this Chapter.



Figure 2.1 Investigating and sampling in field after the 2015-catastrophic landslide occurred at Cao Thang ward, Ha Long city, Quang Ninh province, Vietnam

2.1 Introduction

2.1.1 The Mechanism of Rain-induced Rapid Landslides

Landslides are one of the most dangerous geo-hazards which mainly occur during heavy rainfall. Understanding the initiation mechanism and behavior of the landslide triggered by the extreme rainfall is needed to reduce the landslide risk.

Rainfall infiltrates into two types of slopes and then cause shallow and deep landslides as illustrated in Figure 2.2 (Sassa, et al. 2013).

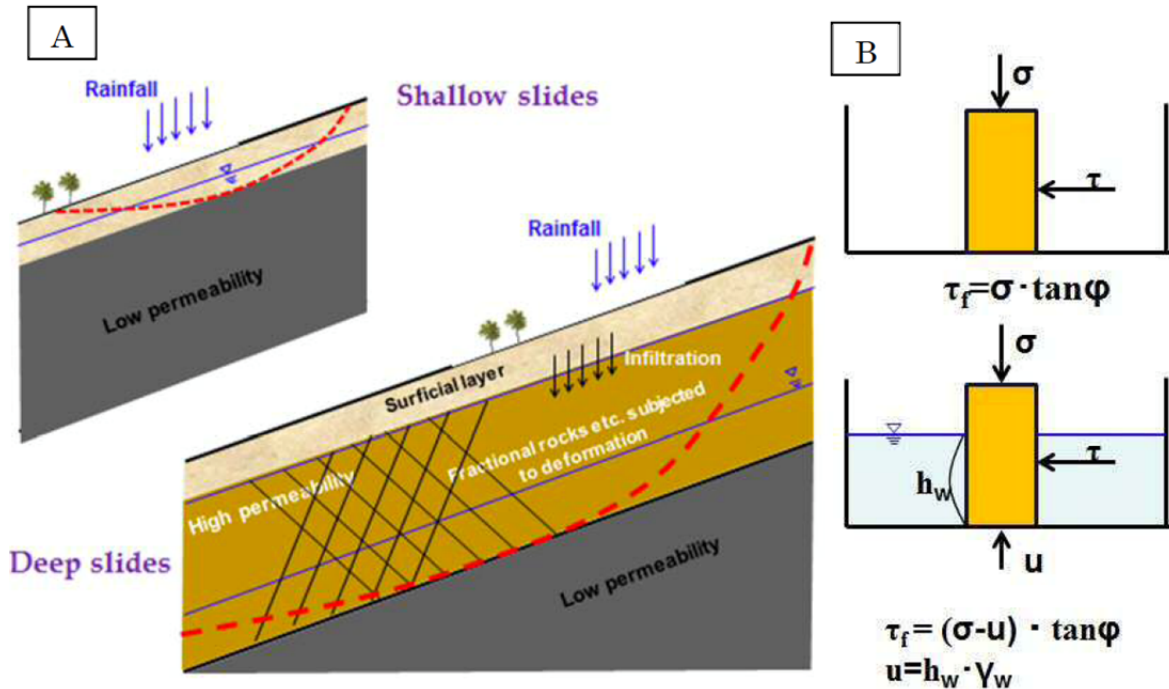


Figure 2.2 Rainfall and groundwater formation and pore-water pressure (Sassa, et al. 2013)

The process starts since the rain water infiltrates into the slope then ground-water flow will appear above the layer with low permeability. Consequently, the shear

strength of the soils or fractured/weathered rocks is decreased leading to shallow and deep landslides.

The effect of ground water rise in a manner is presented in Figure 2.2B as above. Water pressure (u) inside the soil mass is called pore-water pressure and acts on the bottom of the block and its value is (water height) $\times$ (unit weight of water). Water pressure acting on the sides of the block will be cancelled. Hence, the shear stress at failure will be decreased by the value of the pore pressure,

$$\tau_f = (\sigma - u) \cdot \tan \phi$$

Within the soil mass in a slope, pore-water pressure acting on the soil base is different and the value of pressure is NOT (water height) $\times$ (unit weight of water) as Figure 2.2B. The ground-water flow within a soil layer is assumed parallel to the bed rock as illustrated in Figure 2.3. The pore-water pressure (u) acting at the base of the soil layer is $u = z_w \cdot \gamma_w \cdot \cos^2 \theta$

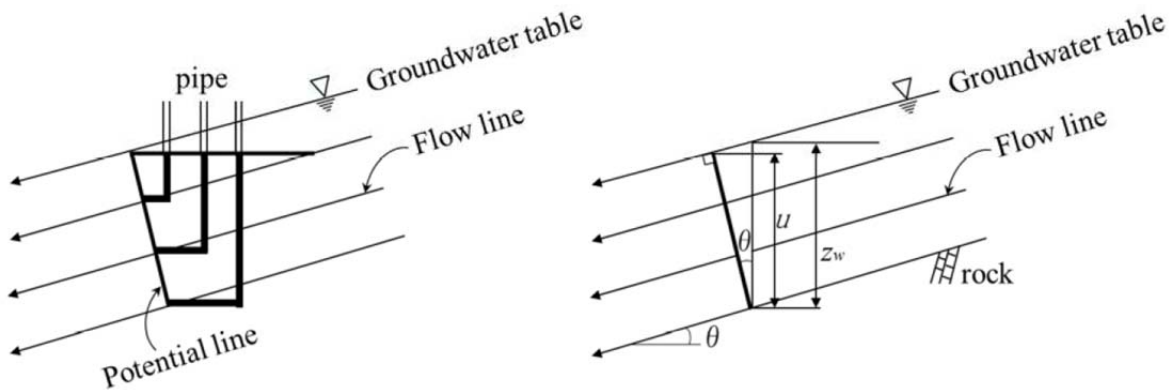


Figure 2.3 Pore-water pressure in the ground water flowing parallel to a slope
(Sassa, et al. 2013)

In a tectonically active area, shear bands with large pores and high permeability may established by tectonic stress. During rainfalls, water flows may be carried by dry channels passing across shear bands. If the permeability of the shear bands is much higher than of the potential landslide mass, high water pressure may act on the base of the landslide susceptible layer (potential sliding surface). The mechanism of pore-pressure acting on the sliding surface can be supplied from an underlying layer is shown in Figure 2.4

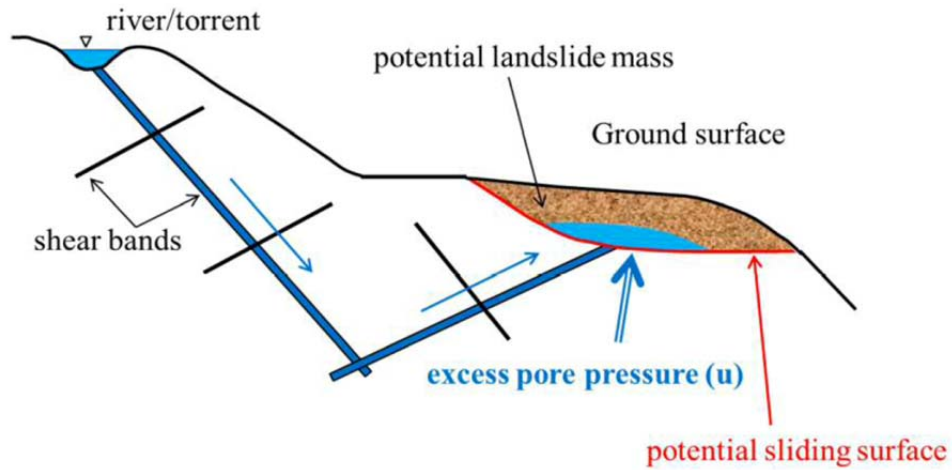


Figure 2.4 Pore-water pressure acting on a potential sliding surface from an underlying layer (Sassa, et al. 2013)

The mechanism of rainfall-induced landslides is illustrated in Figure 2.5 for the cases of pore-water pressure shown in Figure 2.2 and Figure 2.3 (Sassa, et al. 2013).

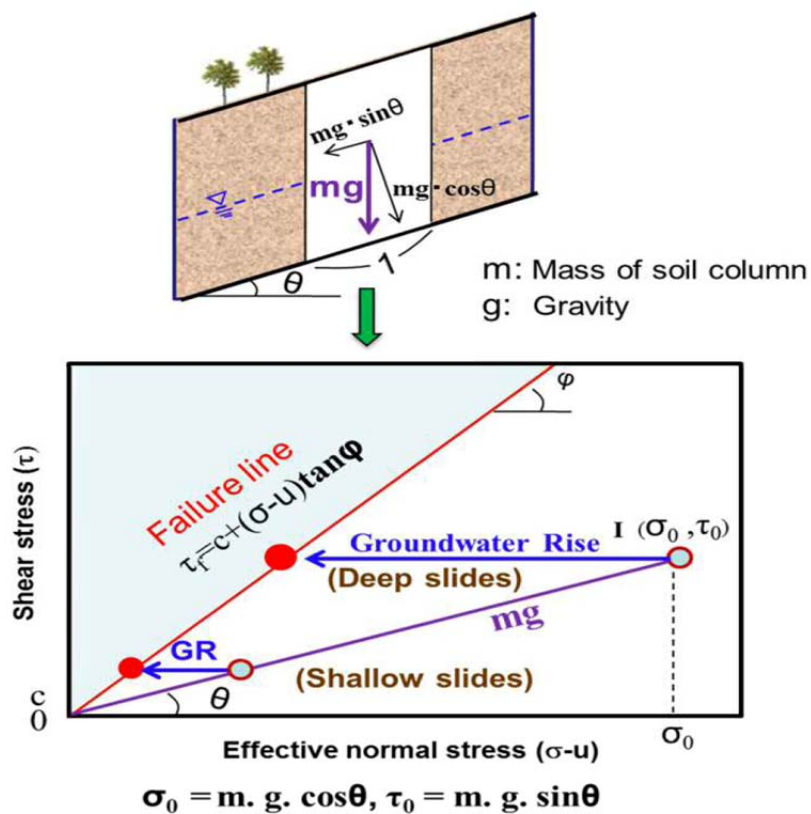


Figure 2.5 Landslide-initiation mechanism due to ground-water rise / pore-water pressure rise (Sassa, et al. 2013)

The base of a soil (weathered rock) layer is assumed to be landslide susceptible layer where the soil column with a unit length along as shown in the upper figure.

The weight of the soil column (W in Figure 2.6) is expressed as mg in Figure 2.5. Here, m is calculated as following:

$$m = \rho \cdot g \cdot Z \cdot \theta \quad (2-1)$$

Where ρ is density of the soil layer.

The initial stress at the bottom on this column is plotted as I in the stress-path figure of normal stress and shear stress (Figure 2.5) in the case of no pore-water pressure acting. The initial stress point is on the line with an inclination of slope angle θ and the length between the origin and the stress point I represents mg (weight of soil column W). Hence, we have normal stress at I is $\sigma_0 = mg \cos \theta$ and shear stress at I is $\tau_0 = mg \sin \theta$.

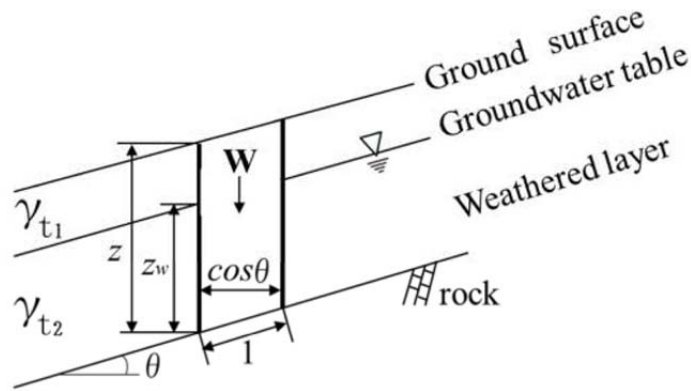


Figure 2.6 Stress acting on the sliding surface within a simple slope (Sassa, et al. 2013).

The stress moves toward to the left when ground-water level and pore-water pressure (u) increases. The failure line is red line and red circle illustrates when shear failure will occur at the stress at failure (Figure 2.5). The shear stress at failure, τ_f is expressed as following:

$$\tau_f = c + \sigma \tan \phi \quad (2-2)$$

Where:

τ_f : Shear stress at failure (shear strength)

σ : Normal stress

c : Cohesion

ϕ : Internal friction angle

2.1.2 Study Area-The 28 July 2015 Rapid Landslide at Ha Long City, Quang Ninh, Vietnam

Landslides are one of the most dangerous geo-hazards in Vietnam which mainly occur during heavy rainfall during tropical typhoons (Tien Bui, et al. 2013, Duc 2013). The Northern mountainous area is one of the most landslide-prone regions in the country (Tien Bui, et al. 2013, Hung, et al. 2016). In particular, on 28 July 2015, around 01:00 local time, a catastrophic landslide occurred at Cao Thang ward, Ha Long city, Quang Ninh province, Vietnam (Figure 2.7). The landslide claimed 8 lives and destroyed 3 houses.

Ha Long city is located in the center of Quang Ninh province where there is complex and diverse coastal topography including hills, delta, and islands. It is a highly urbanized and rapidly growing city (population 221,580 in 2010). Much of the population live in hilly terrain prone to landslides and debris flows. The North and North East are covered by hills, containing the 70% city's area. Altitude ranges from sea level to 504 m.



Figure 2.7 Location map of Ha Long landslide (from Google Earth)

Ha Long has a tropical coastal climate with two seasons. A summer season lasts from May to October; the winter from November to April. The average annual precipitation is 1800 mm with 80 - 85 % of the annual average falling in summer, especially in July and August. A geological map of part of Ha Long city (Figure

2.8) illustrates that the landslide area consists of Triassic age conglomerates, gritstones, sandstones and thin lenses of coal.

The Bai Chay weather station in Ha Long city recorded a total rainfall of 387 mm between August 27th and 28th. Especially, the cumulative rainfall in the 12 hours between 19:00 hr, 27 July and 07:00 hr, 28 July was 296 mm. The main triggering factor of the Ha Long landslide was this intense rain falling in the short time period. Total rainfall measured at a number of gauging stations in Quang Ninh province during 26-30 July showed the highest readings in the last 40 years (CCNDPC 2015). The cumulative rainfall from 19:00, 27 July, until 07:00, 28 July was 296 mm at Bai Chay weather station in Ha Long city (CCNDPC 2015).

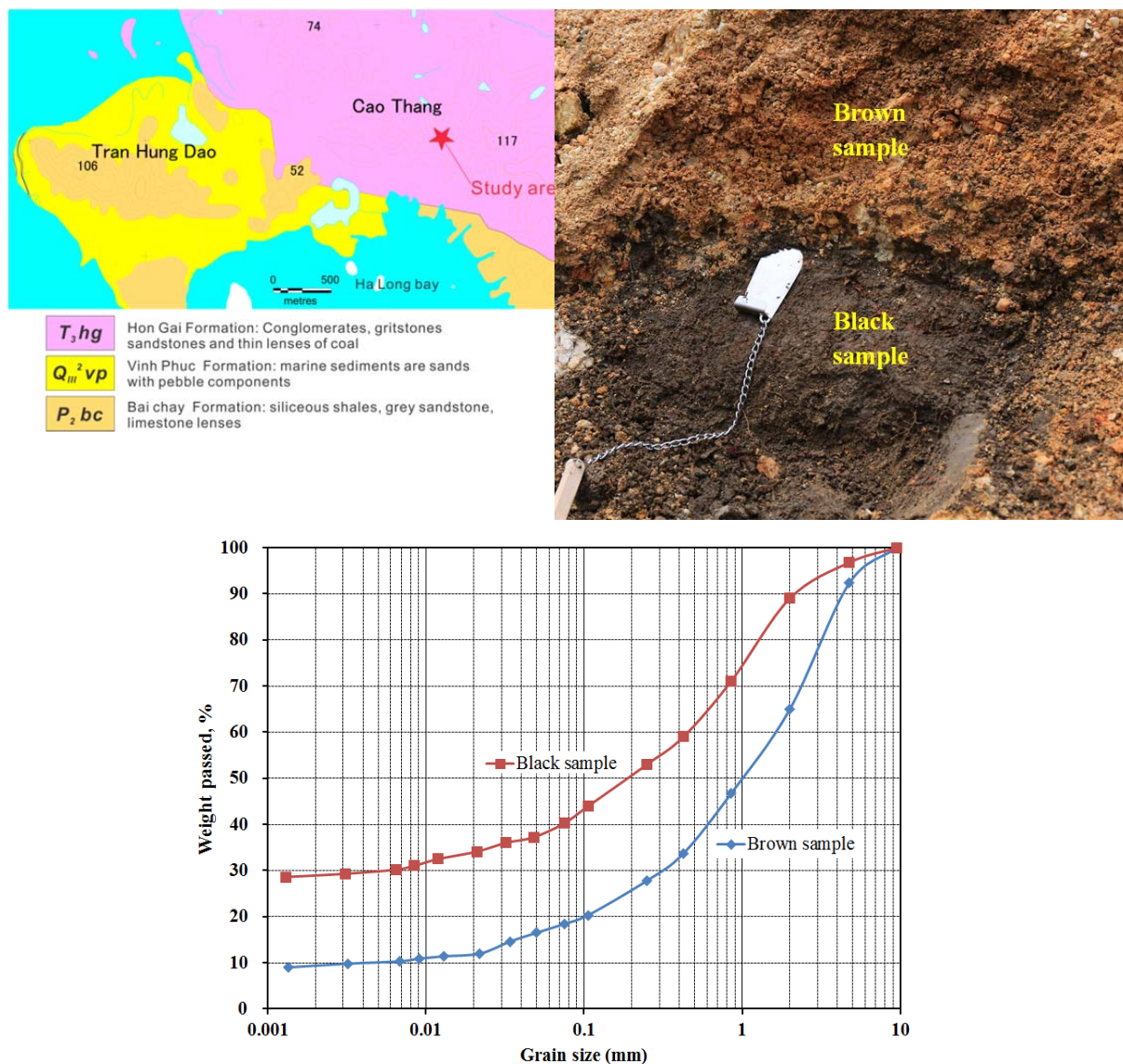


Figure 2.8 Geological map of study area and sampling

2.2 Methodology

2.2.1 SLIDE Model

To simulate rainfall induce landslide, pore-water pressure changes need to be estimated. Based on the research of Montarasio, et al. 2008 and Liao, et al. 2010a, the Slope-Infiltration-Distributed Equilibrium (SLIDE) model can estimate pore-water pressure from rainfall intensity using the simple assumption that the water depth in a column in an infinite slope is changed by the balance of rainfall infiltration (assuming all rain infiltrates into the ground) and the drainage outflow from the column. Figure 2.9 presents the concept of the SLIDE Model.

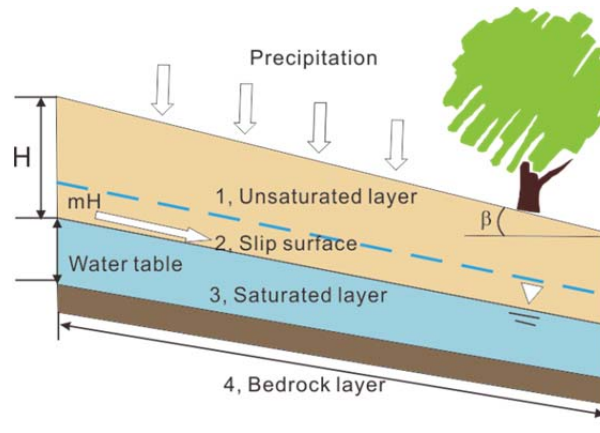


Figure 2.9 Schematic illustration of water infiltrating an infinite slope (from Liao, et al. 2010b)

Rainfall penetrates into the soil layer and a saturated layer is formed above the bedrock, allowing, groundwater to flows through the slope. When the water supply from the ground surface exceeds the ground-water discharge from the saturated layer, the ground-water level and associated pore-water pressures will increase. Conversely, the ground-water level (pore-water pressure) will decrease when the infiltration of rain is less than the ground-water discharge from the saturated layer. According to Liao, et al 2010 pore pressure (ΔU) acting at the sliding surface can be expressed as following:

$$\Delta U = m.H.\gamma_n.\cos^2 \beta \quad (2-3)$$

And, changes in the ratio of groundwater layer to the soil layer pressure (m) is calculated as following:

$$\left\{ \begin{array}{l} m_1 = 0 \\ O_t = K_t \cdot \sin \beta \cdot m_t \cdot H \cdot \cos \beta \cdot \Delta t \\ \Delta m_t = \frac{(I_t - O_t)}{n \cdot H \cdot (1 - S_r)} \\ m_{t+1} = m_t + \Delta m_t \end{array} \right. \quad (2-4)$$

Where ΔU is pore-water pressure, m is the ratio between the depth of the soil layer and the depth of the groundwater layer, t is time, Δt is time interval, m_1 is initial value of m , and m_t is calculated at each time step. O_t represents the water outlet of a finite portion of a slope of finite length L . I_t is rainfall intensity, and K_t is average permeability of the soil layer, H is landslide depth, n is porosity; β is slope angle; S_r is degree of saturation (Liao, et al. 2010).

SLIDE model is a simplistic model that neglects certain effects such as transpiration from trees and vegetation, and the initial unsaturated infiltration process, but provides an appropriate model for use in computer simulation.

We used rainfall intensity at the Bai Chay weather station to study rainfall induce the Ha Long landslide and pore-water pressure response to the rainfall was calculated using the SLIDE model. We used an average soil permeability of 0.00362 m/s, a saturation of 83%, a soil thickness of 6 m, slope angle of 16°, and porosity of 41%.

The excess pore-water pressure ratio (r_u) during the heavy rainfall period was calculated as the landslide trigger factor in a LS-RAPID simulation. It was expresses as: $r_u = \Delta u / \sigma$

Where: σ is total normal stress acting on the potential sliding surface $\sigma = H \cdot \gamma \cdot \cos^2 \beta$ (γ is unit weight of soil $\gamma = 19.2 \text{ kN/m}^3$).

Figure 2.10 shows the relationship between rainfall intensity and pore-water pressure ratio from 0:00 hr on 26 July 2015. The maximum pore pressure ratio is 0.41.

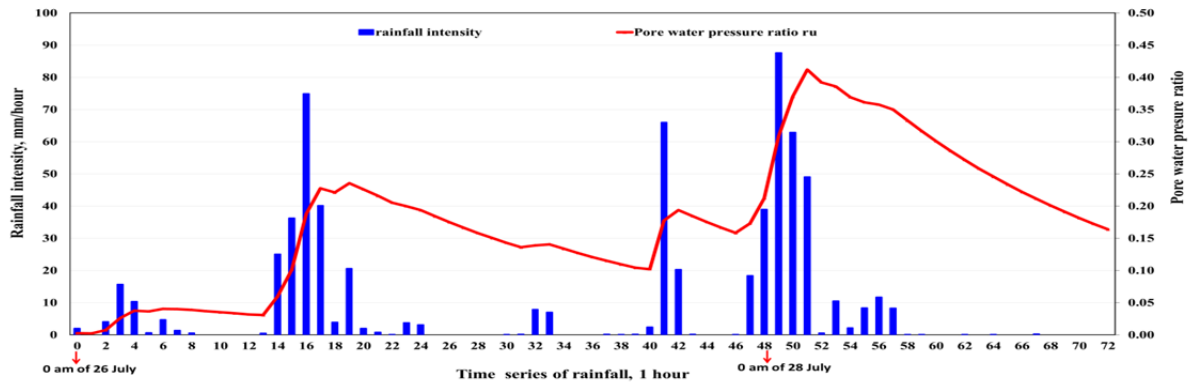


Figure 2.10 Pore-water pressure calculation by the SLIDE model from the rainfall record monitored at the Bai Chay weather station for each 1 hour from 0:00 hr on 26 July 2015

2.2.2 UAV method for landslide mapping

Unmanned Aerial Vehicles (UAVs), also known as Unmanned Aircraft Systems (UAS) is a new method for monitoring and assessing landslides. UAVs method using a remotely piloted aircraft is able to collect ultra-high-resolution imagery from which Digital Terrain Model (DTM) of a landslide is created. By using such models, it is possible to generate volumes, areas, cross sections and contour lines in a very short time (Marijan, et al. 2016).

In this study, we used a Phantom 4 by DJI which has four propellers and is equipped with a 4K video at up to 120 frames per second and capturing 12 megapixel photos and the new 5,350 mAh DJI Intelligent Flight Battery to provide flight at up to around 28 minutes. The flight planning was designed by using the DJI go software compatible with the UAV. The longitudinal and transversal overlaps more than 70%. After taking the photos from UAV, the photo processing, building dense point cloud, building mesh (3D polygonal model), building digital elevation model and orthophoto map take place automatically by Agisoft PhotoScan.

Figure 2.11a and Figure 2.11b present a UAV image of the affected area and a longitudinal section of the Ha Long landslide. This section is based on a digital surface model (DSM) generated from post-landslide UAV images and a pre-landslide 5-m topographical map. The maximum landslide depth is 6.0 m (vertical).

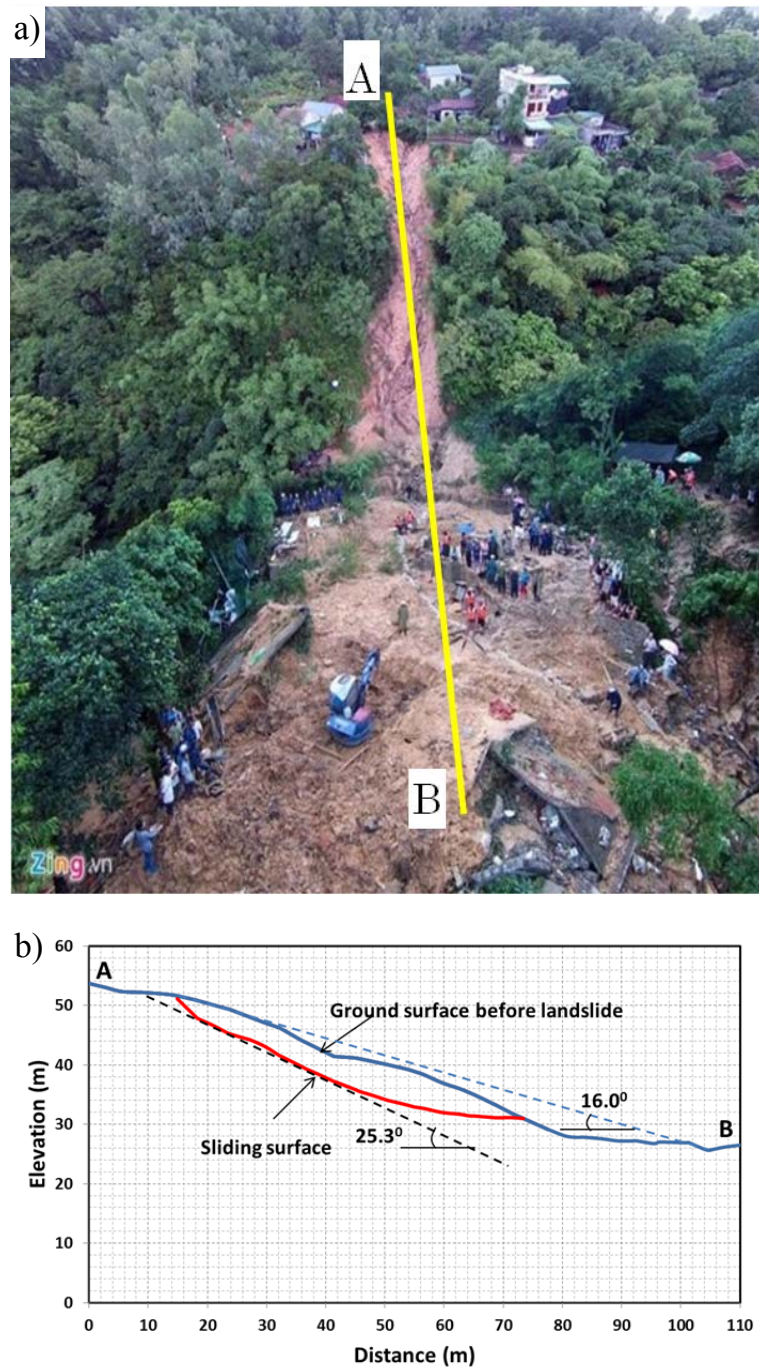


Figure 2.11 UAV image (Modified from Zing) (a) and A-B longitudinal cross section of Ha Long landslide (b)

2.2.3 Ring-shear Simulator for Landslides Triggered by Rainfall

Samples taken from landslide area were tested in a ring-shear box (Sassa, et al. 2004a). The normal and shear stresses acting on the potential sliding surface were reproduced in the shear box. When stresses became high enough to trigger sample deformation, a shear surface developed within the shear box, and the rotary lower half of the shear box started displaced. During the shearing process, the ring shear

apparatus measured excess pore-water pressure, mobilized shear resistance and shear displacement. Two tests were conducted on Halong sample. The test program is present in Table 2.1.

Table 2.1. Test program

Sample	Test	Consolidation stage		Shear stage		
		Normal stress (kPa)	Shear stress (kPa)	Shear stress increment rate (kPa/sec)	Pore-water pressure increment rate (kPa/sec)	Drainage
Brown sample	Undrained monotonic shear stress-controlled test	95	0	0.5		Undrained
Brown sample	Pore-water pressure-controlled test	95	45		0.4	Undrained

An undrained monotonic shear stress-controlled test was conducted at a normal stress of 95 kPa. This corresponds to the landslide initiation in the field. Normal stress was increased to 95 kPa at a rate of $\Delta\sigma=1$ kPa/s in the drained condition. After consolidation, the shear box was then changed to the undrained condition, and shear stress was increased at a rate of $\Delta\tau=0.5$ kPa/s. When the effective stress path reached the failure line, the sample deformation and pore water pressure was recorded until 10 m of shear displacement had been achieved.

The Ha Long landslide was triggered by heavy rainfall which can be simulated using the ring-shear apparatus. In order to observe the effect of pore-water pressure generation carefully, the initiation of the slope failure was simulated. Firstly, the sample was saturated ($B_d=0.98$), then consolidated to 95 kPa normal stress and 45 kPa shear stress in a drained condition.

2.2.4 Integrated Simulation Model LS-RAPID for the Initiation and Motion of Landslides Triggered by Rainfall

LS-RAPID is an integrated computer model that can simulate the initiation and motion of a landslide using soil parameters obtained from the ring-shear results. This simulation model was developed from the geotechnical model for the motion

of landslides (Sassa, et al. 2010) its improved simulation model and new knowledge obtained from a new dynamic loading ring shear apparatus (Sassa, et al. 2004a, 2010).

It is difficult to simulate both landslide initiation and motion in a single model. Thus, the state from the initiation of the landslide to its occurrence was simulated using slope stability analysis, while the motion of the landslide was simulated using landslide run out analysis. Ring shear test results were input into LS-RAPID as the landslide's dynamic parameters.

The pore-water pressure ratio calculation by the SLIDE model from the rainfall record monitored at the Bai Chay weather station was then input into LS-RAPID as a graph of triggering factor.

2.3 Sampling, Testing and Results

2.3.1 Sampling

On 10 September 2015, Japanese and Vietnamese researchers investigated the landslide area. Two samples were taken from the landslide head scarp as shown in Figure 2.8 and sent to the International Consortium Landslide (ICL) laboratory in Kyoto University for testing. The soils in the landslide source area were characterized by a series of standard geotechnical tests (Table 2.2). There is a brown layer above black layer. The materials are weathered gritstone including sand and silt. The black layer contains more fines than the brown layer. From the grain size distribution (Figure 2.8), the brown sample had the particles with the mean diameter $D_{50}=1.06$ mm, uniformity coefficient $U_c=363.8$, the grains ranging from sand to clay size. It is sand with $>12\%$ fines. The black sample had the mean diameter $D_{50}=0.20$ mm. The density of brown and black samples were 26.5 and 26.7 kN/m³. Permeabilities of the black and the brown samples are 4.50×10^{-7} and 3.62×10^{-3} m/s, respectively. Due to permeability change between the two layers, the sliding surface may have occurred at the base of the brown sand layer. The brown sand sample was used in the ring-shear tests to study the initiation mechanism and motion behavior of the landslide at Ha Long triggered by heavy rainfall.

Table 2.2. Basic geotechnical test results of the two samples

Sample	Moisture (%)	Unit weight (kN/m ³)	Density (kN/m ³)	Permeability m/s
Black sample	26.6	18.7	26.5	4.50×10^{-7}
Brown sample	21.2	19.2	26.7	3.62×10^{-3}

2.3.2 Un-drained Monotonic Shear Stress-controlled Test

Figure 2.12 presents the stress paths and time series data for the brown sand sample. The measured steady-state shear resistance was 6 kPa. The friction angle of the peak failure line and friction during motion were 38.0° and 35.6° , respectively.

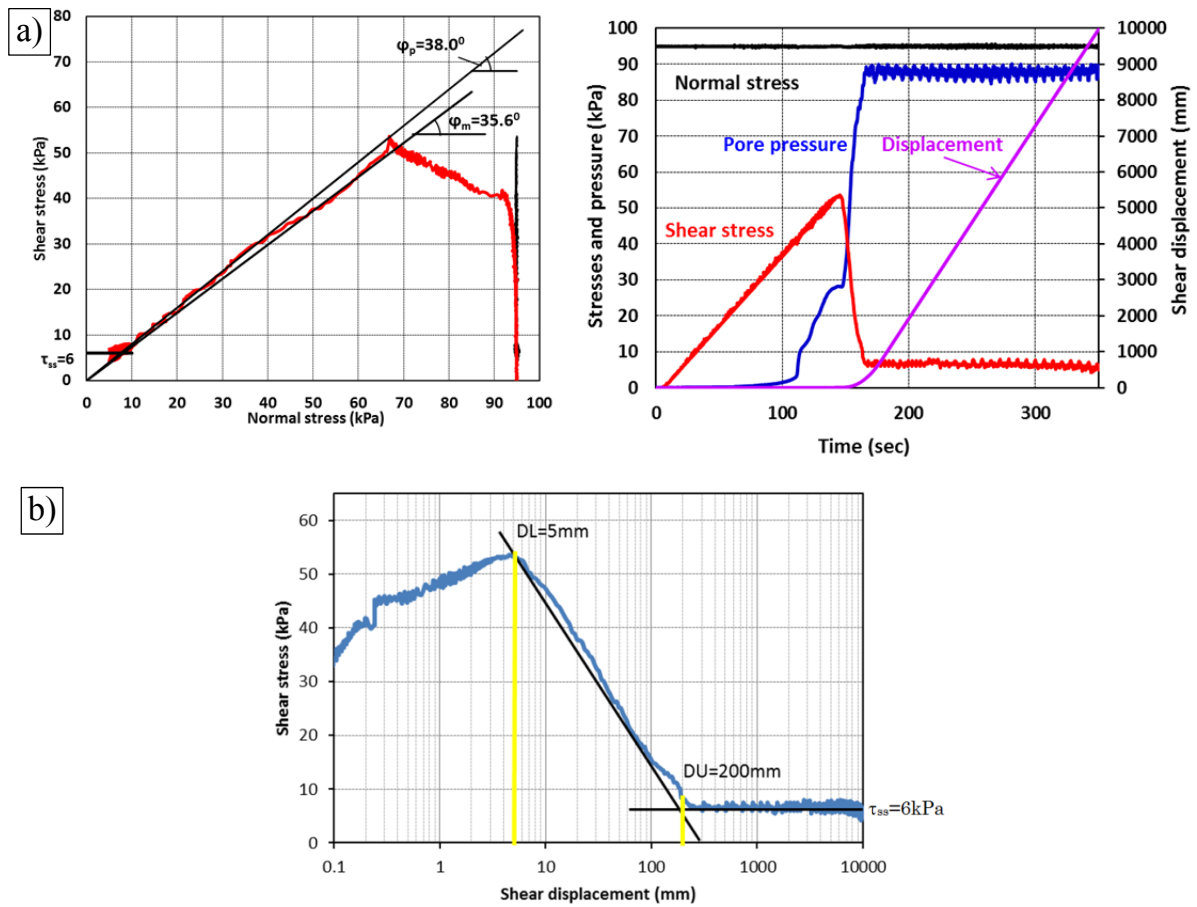


Figure 2.12 Undrained monotonic stress-controlled test on the brown sand sample ($B_d=0.95$, normal stress=95 kPa, $\Delta\tau/s=0.5$ kPa/s). Stress path and time series data (a), Relationship between shear stress and shear displacement (b).

2.3.3 Pore-water Pressure-controlled Test

The preparatory stage was to reproduce the initial stress on the slope (the corresponding slope angle is 25.3°), and is shown as a black line in Figure 2.13.

In order to simulate the pore-pressure induced landslide process, the water pressure was gradually increased at a rate of $\Delta u = 0.4$ kPa/s up to 95 kPa. Failure occurred at a pore-water pressure of 32 kPa at a pore-water pressure ratio $r_u = 32/95 = 0.34$. The friction angle at failure was 36.3° . To monitor the water pressure during shearing, the pore-water pressure supply valve was closed after failure occurred. Figure 2.13 shows the stress path going down the failure line during motion with a friction angle of 32.0° during motion. After closing the pore-water pressure supply valve, the pore-water pressure was quickly generated to close the normal stress during shearing. The shear resistance is reduced and reached steady state with high speed of shearing. This result indicates that the sliding-surface liquefied as has been observed by other researchers (e.g. Sassa, et al. 2004, 2010, Wang, et al. 2005, Okada, et al. 2007, Dang, et al. 2016). The sliding-surface liquefaction is key concept of landslide dynamic. Grains in the shear zone were crushed during shearing and a high pore-water pressure is generated by reduction of volume (Sassa, et al. 2004, 2010, Okada, et al. 2007). This test may have reproduced the rapid and fluidized motion of Ha Long landslide.

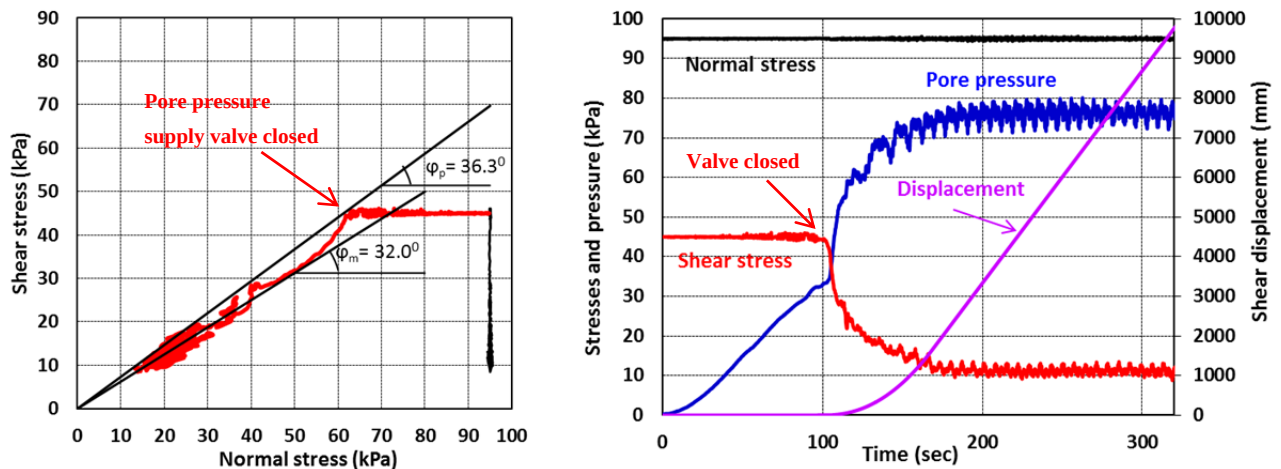


Figure 2.13 Pore pressure-controlled test on the brown sand sample ($B_d=0.98$)

2.4 Application of LS-RAPID and SLIDE Model to the 2015 Ha Long Landslide

2.4.1 Triggering Factors

The amount of heavy rainfall needed to trigger the 2015 Ha Long landslide was calculated using the SLIDE model. This pore-water pressure ratio was then input into LS-RAPID as a graph of triggering factor.

2.4.2 Landslide Dynamics Parameters

The parameters used in the computer simulation are listed in Table 2.3.

Table 2.3. Parameters used in LS-RAPID simulation

Parameters	Value	Source
Steady state shear resistance (τ_{ss} , kPa)	6	Test data
Lateral pressure ratio ($k=\sigma_h/\sigma_v$)	0.5 – 0.7	Estimated
Friction angle at peak (ϕ_p , degree)	38.0	Test data
Cohesion at peak (c, kPa)	5	Estimated
Friction angle during motion (ϕ_m , degree)	35.6	Test data
Shear displacement at the start of strength reduction (DL, mm)	5	Test data
Shear displacement at the start of steady state (DU, mm)	200	Test data
Pore pressure generation rate (B_{ss})	0.3–0.95	Estimated
Total unit weight of the mass (γ_t , kN/m ³)	19.2	Test data
Unit weight of water (γ_w , kN/m ³)	9.8	Normal value
Triggering factor		
Pore-water pressure ratio in the potential shear zone (r_u) during rain	0–0.45	Estimated from SLIDE model

The parameter information was described as follows:

- (1) The steady-state shear strength, the peak friction angle, the friction angle during motion ($\phi_p=38.0^\circ$) was obtained from the undrained stress-controlled test.
- (2) The shear displacement to achieve shear-strength reduction was estimated from the undrained stress-controlled test. The critical shear displacement for the start of strength reduction (DL) and the start of steady state (DU) were obtained from shear stress and shear displacement graph.
- (3) Lateral pressure ratio k: 0.5 – 0.7. The authors assumed the lateral ratio to be 0.5 in the top of the slope, 0.6 in the landslide body and 0.7 in the lower part.
- (4) Pore-pressure generation rate B_{ss} was 0.8–0.9 in the landslide area, and 0.3 outside the landslide area. It was 0.95 in the marsh area (inside of yellow line in 0a)
- (5) The unit weight of the brown sand sample was 19.2 kN/m³.

2.4.3 Simulation Result of the Landslide

The simulation result of the Ha Long landslide is presented in 0. The pink line is the calculated sliding mass distribution area. Then, the pore-water pressure ratio was increased and local failure started from the middle of the slope when r_u reached 0.34 at 01:30':00" (28 July). At 01:55':39", r_u reached 0.36 and the whole landslide mass was formed and moved down. At 01:55':49", the landslide mass reached a maximum velocity of 13.3 m/s. At 02:10':19", movement had ceased. Using the image taken by UAV (0f) as the real case, the landslide movement in the computer simulation is similar to it. Based on information from local inhabitants, the Ha Long landslide took place around 01:00 local time on 28 July. The landslide is modeled to have occurred at 01:30':00" (0b). Allowing that the rainfall input into the model has an hourly resolution, the estimated time of landslide occurrence from the LS-RAPID simulation is remarkably close to the actual occurrence time.

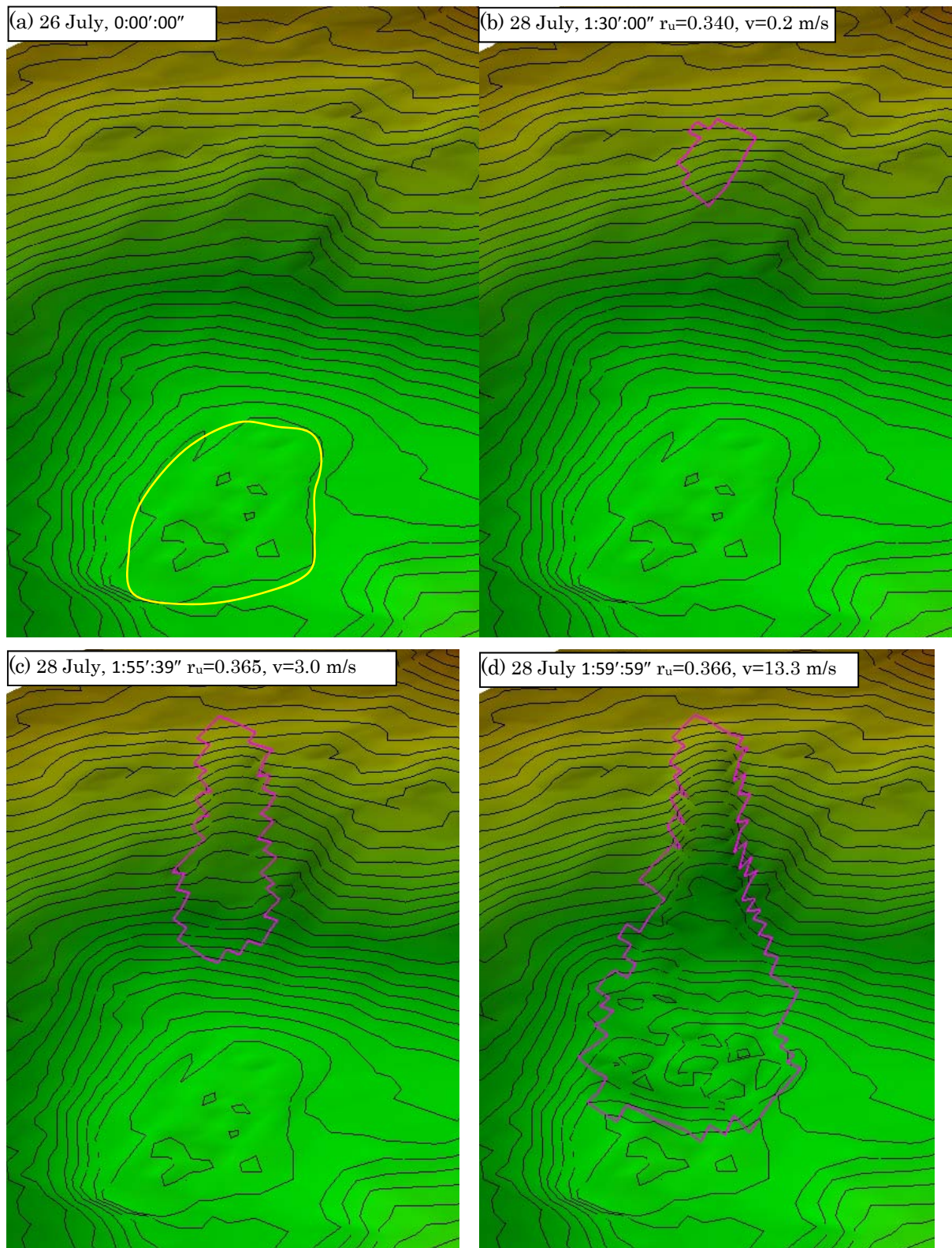


Figure 2.14 Simulation result of Ha Long landslide

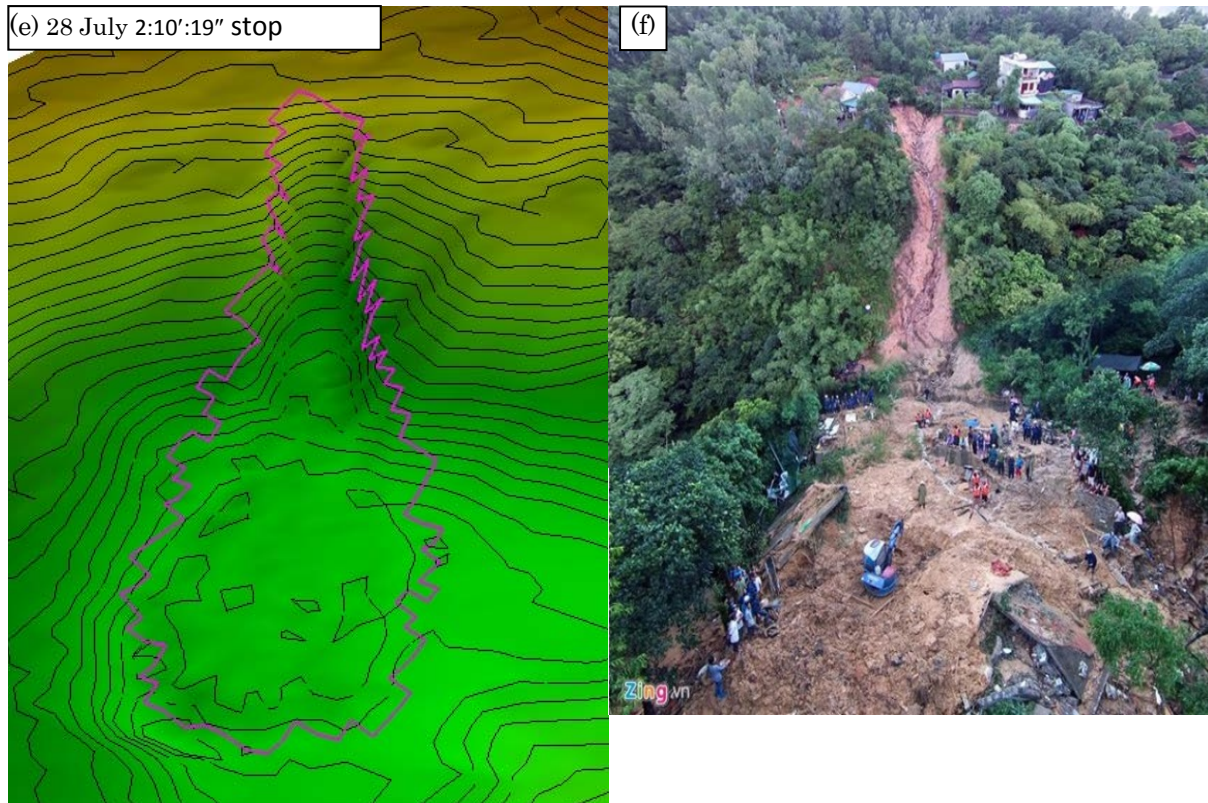


Figure 2.14 (continued)

2.5 Summarization

Rainfall-induced landslides and debris flows resulted in a great disaster in Ha Long. An explanation the initiation mechanism and behavior of the Ha Long landslide is presented in this chapter. The pore-water pressure-controlled test for simulating the initiation of landslide can suggested that Ha Long landslide could be occurred by increasing pore-water pressure until a pore-water ratio of 0.34. We examined these results in relation to the rainfall event which was known to have triggered the event.

Chapter 3 Study of the Mechanism of Earthquake-induced Rapid Landslides

As mentioned above, the project “Development of landslide risk assessment technology along transportation arteries in Vietnam” was implemented in the period of 2011 to 2016 that was the successful corporation by both Japanese and Vietnamese experts. Main achievements acquired by the WG3-Testing group, where the candidate was the Vietnamese side leader, were defined as the creation of new the new high-stress undrained ring-shear apparatus, ICL-2 as well as the application of UAV technology for landslide studies. These were presented in the report of “Recent development of the new high stress undrained ring shear apparatus and its application” by Lam Huu Quang (the candidate) et. al. in 2014.

In April 2016, the occurrence of a strong earthquake caused a large number of landslides damaged to people and properties in Kumamoto, Japan. A team led by Prof. Kioji Sassa, Prof. Hiroshi Fukuoka with other Japanese and Vietnamese experts conducted an investigation, sampling and taking aerial photos by UAV method. Then, two joint WGs tested samples and analyzed data to simulate the mechanism of initiation and motion of Kumamoto landslides. As a leader member, the candidate’s assignment was making analysis based on ring shear test results and simulation the earthquake-induced landslide mechanism utilized LS-RAPID. The main content of this research was published in the Landslide Journal titled as “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)” and is presented in this Chapter.

3.1 Introduction

3.1.1 The Mechanism of Earthquake-induced Landslides

Strong earthquakes usually lead to the occurrence of a large number of landslides, causing great devastating disasters. Earthquake-induced landslides occur suddenly, and the process is complex, which is assumed as dynamic behavior from the

physical point of view. The dynamic process has been analyzed quantitatively or semi-quantitatively (ShouBiao, et al. 2013).

Steep slopes and gentle slopes are where earthquake-induced landslides may occur on (Sassa, et al. 2013). k is called the seismic coefficient which is the ratio of the seismic acceleration (a) and gravity (g), namely $k=a/g$. And $k.mg$ is amount that expresses the loaded stress when an earthquake appears and a seismic acceleration is loaded.

The direction of the seismic stress increment is expressed as $(\alpha + \theta)$ where θ is slope angle and α is the direction of seismic force from the vertical direction. The effective stress path during the earthquake is not the same direction of the total stress path.

Seismic stress is a dynamic stress. It is different from gravity (permanent) and pore water pressure (relatively slow change). The case of the initial stress and the seismic stress reaching the failure line is expressed in Figure 3.1. The effective stress path shifts from I to A_f if pore-water pressure is generated during seismic loading.

The Figure 3.2 expresses the case of landslide initiation mechanism by seismic loading in addition to water-pore pressure rise. Seismic stress includes a shear-stress ($\Delta\tau$) and a normal-stress component ($\Delta\sigma$). In this case, the effective seismic stress increment is mostly vertical because the normal-stress increment will be nullified by the generated pore-water pressure when the sliding surface is fully saturated.

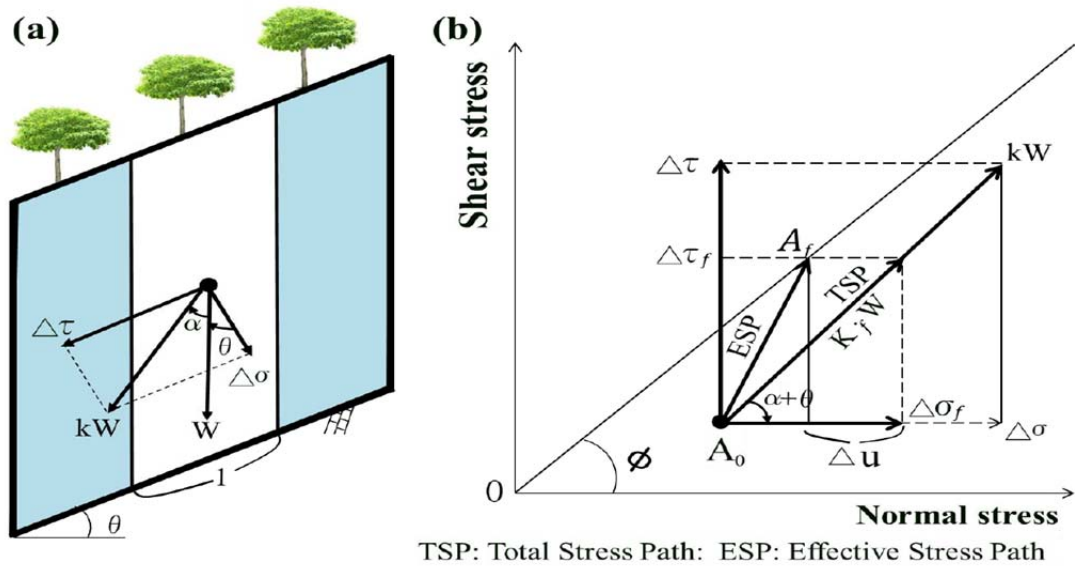


Figure 3.1 Landslide-initiation mechanism due to effect of earthquake (Sassa et al. 2013)

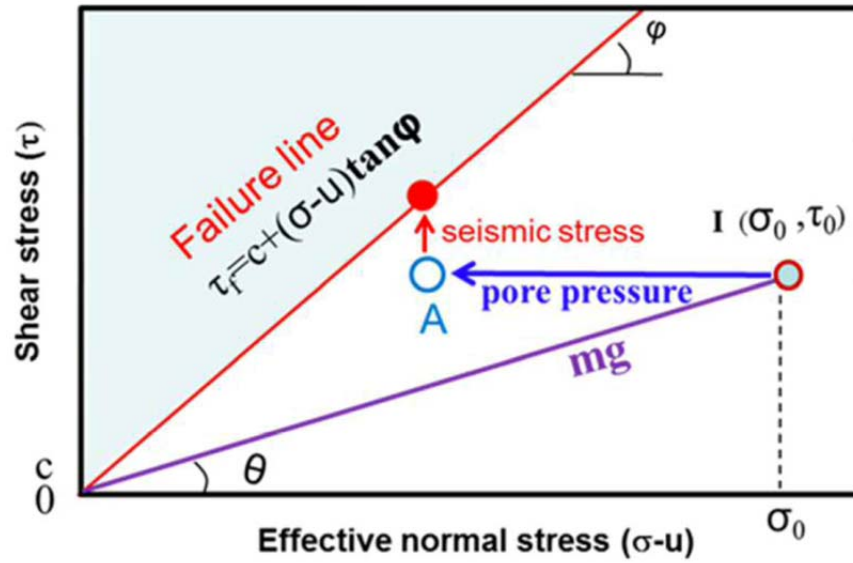


Figure 3.2 Landslide-initiation mechanism by the combined effect of earthquake & pore-pressure rise (Sassa et al. 2013)

3.1.2 Study Area-Landslides induced by the 16 April 2016 Kumamoto Earthquake, Japan

The 2016 Kumamoto earthquakes were a series of earthquakes, including two main shocks which occurred beneath Kumamoto City, Kumamoto Prefecture, Kyushu Region, Japan. A M6.5 earthquake occurred at 21:26 JST on April 14, and a M7.3 earthquake struck at 01:25 JST on April 16 (National Research Institute for Earth Science and Disaster Resilience, NIED). The near-fault strong ground motion was reported by Furumura 2016. The two earthquakes killed at least 49 people and injured about 3,000 others. More than 44,000 people were evacuated from their homes due to the disaster. These two events generated most of the building damage and many of the landslides in the Kumamoto area. According to Japan's Ministry of Land, Infrastructure, Transport and Tourism, at least 97 landslide locations were confirmed in the Aso area. Among these earthquake-triggered landslides, the largest landslides were two very substantial slope failures (Figure 3.3). One was located on the National Road 57 and destroyed an important bridge. The other occurred near the Aso volcanological laboratory of Kyoto University and destroyed seven houses.

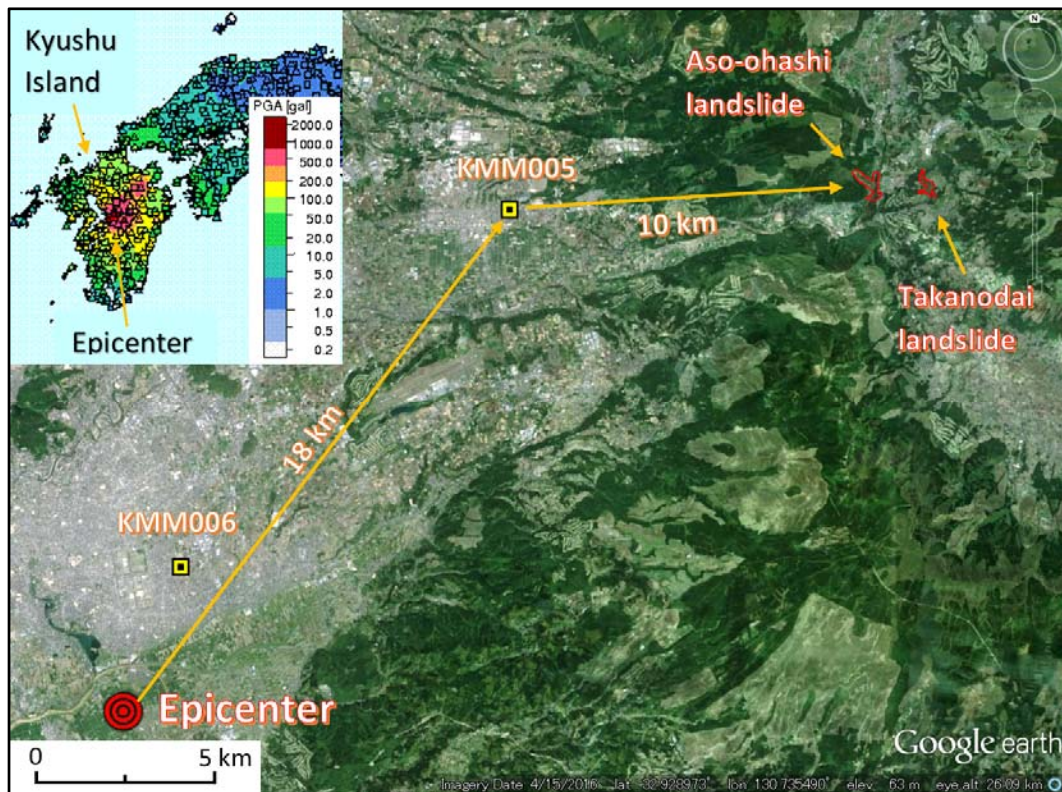


Figure 3.3 Monitored peak acceleration of the M7.3 Kumamoto earthquake (from the National Research Institute for Earth Science and Disaster Resilience, NIED) and locations of the epicenter, earthquake record stations, and the two studied landslides (from Google Earth images)

An overview of the Takanodai landslide occurred on 21 May 2016 (more than a month after the event) is presented in Figure 3.4. This area includes three landslide blocks on the hill-slope below the Aso volcanological laboratory of Kyoto University in Minamiaso village, Kumamoto Prefecture. The largest one destroyed seven houses of the Takanodai housing complex (yellow zone, identified by comparing Google Earth images before and after the event) and killed five people (Ministry of land, infrastructure, transport and tourism). After triggering by earthquake, this landslide moved at least 150 meters.

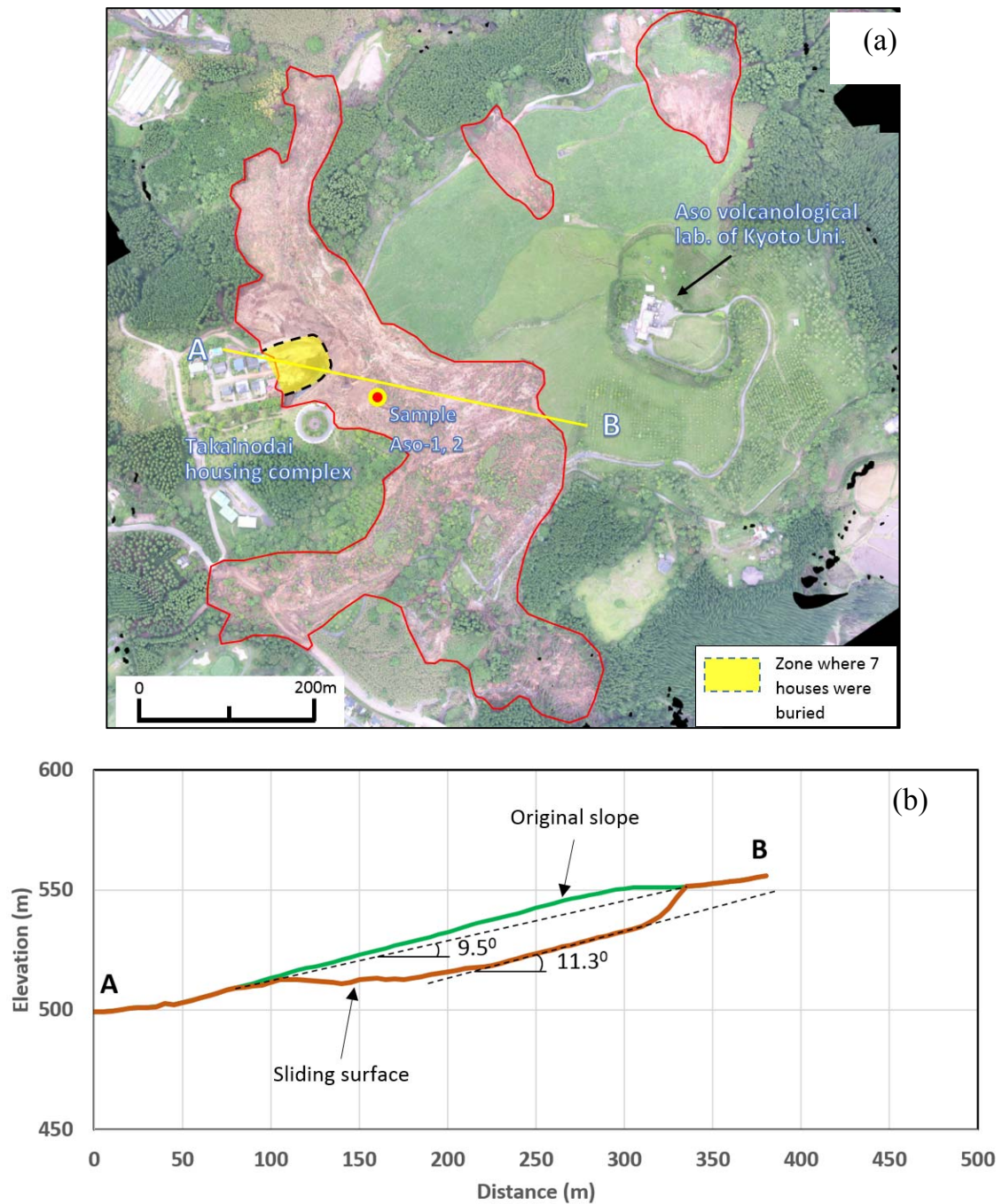


Figure 3.4 Plan and central cross section of the Takanodai landslide area, Minamiaso village, Kumamoto Prefecture (the map was made from UAV images using Agisoft PhotoScan software).

Kumamoto prefectural government had created landslide hazard maps based on national government standards for landslide prevention using the following criteria: steep areas at least 5-meters-high with a slope of 30 degrees or more; areas below a rapid mountain stream that has formed an alluvial fan; areas where landslides have occurred or are at risk of occurring (The Mainichi newspapers 2016). The slopes in the Takanodai housing complex area did not meet even one of these criteria (the

slope in this area is less than 30 degrees and there was no evidence of past landslides) so this area was not designated as a landslide hazard zone. In addition, the prefectural government explained that these areas were not geologically shaped as places prone to landslides. The reasons and mechanism of this landslide will be examined in the later part of this research based on the analysis of collected data, the seismic loading test, and computer simulation model (LS-RAPID).

A central longitudinal section A-B through the Takanodai landslide is presented in Figure 3.4b based on field investigation, the digital surface model (DSM) generated by UAV photos after the landslide and the 5 m topographical map before the landslide. This landslide moved with a slope angle of 11.3 degrees and an average apparent friction angle of 9.5 degrees and a maximum depth of around 20 m. Hence, this landslide can be classified as a gentle sloping, moderately shallow landslide (International Consortium on Landslide 2016).

Figure 3.4, the red lines show the borders of the landslides, the yellow lines (A-B) is the central cross section line. In Figure 3.4b, the green line presents the ground surface before the landslide and the brown line presents the sliding surface with an inclination of 11.3 degrees.

The second landslide is situated at the western tip of the caldera of Mount Aso. It was named after the 200-m long Aso-ohashi bridge that formerly spanned the 80-m deep gorge of the Kurokawa River before it was destroyed by the landslide on April 16 during the magnitude 7.3 earthquake.

This area is also characterized by soft ground composed of weathered volcanic cohesive soil (Geological map display system of Geological Survey of Japan, AIST). In order to obtain the soil characteristics, and to compare with the samples from the first landslide area, two similar soil samples were taken on the left-hand side of the landslide body (seen from the upper slope to down slope in Figure 3.5a). The soil layers at this sampling location were visually similar to those at the first sampling location and also varied from black color (lower layer) to brown color (upper layer).

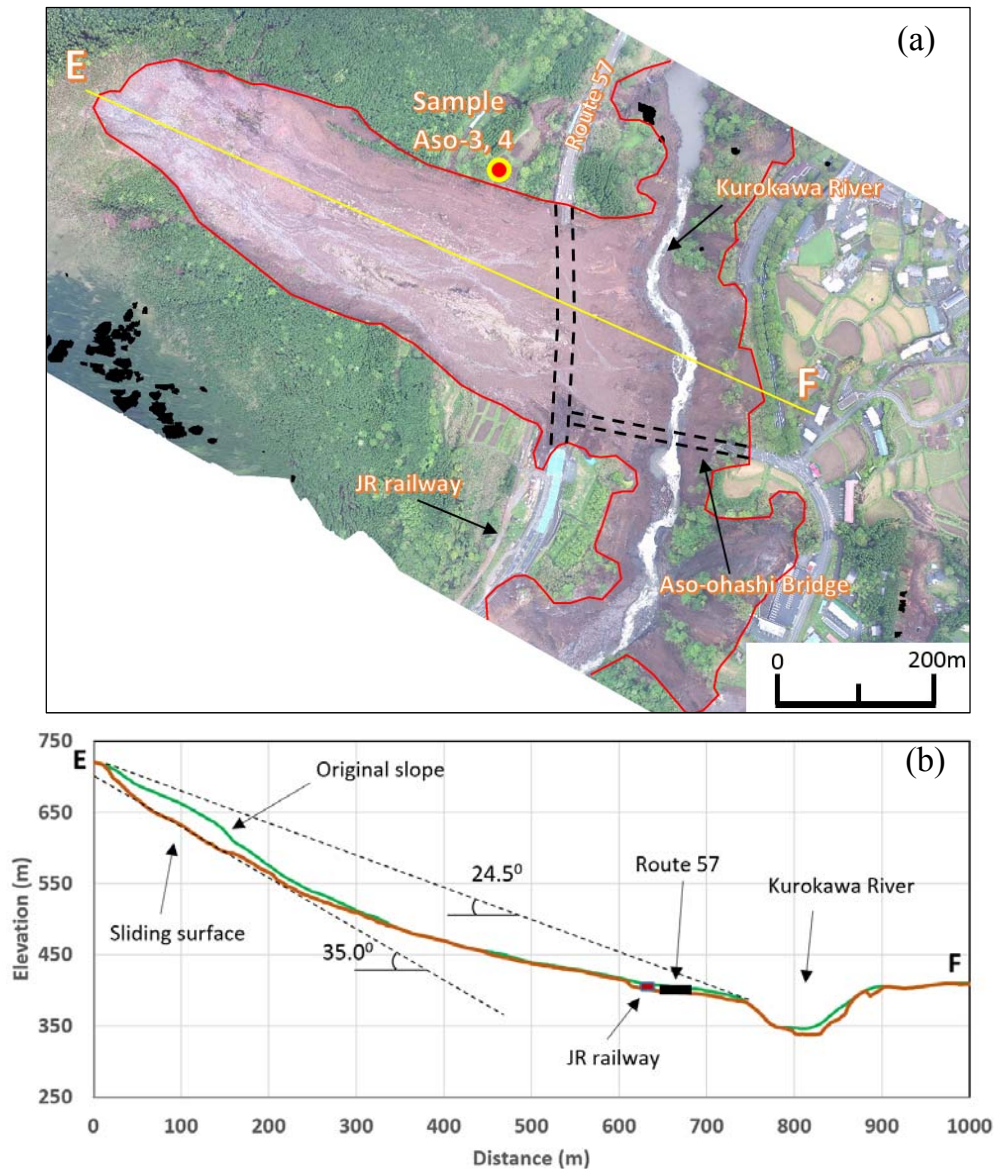


Figure 3.5 Plan and section of the Aso-ohashi landslide (the plan is an orthorectified image made from UAV images with Agisoft PhotoScan software)

Figure 3.5b shows the longitudinal cross section of the Aso-ohashi landslide (line E-F). Green line is the section before the landslide and the red line is the section after the landslide. The average slope angle of the sliding surface was 35.0 degrees and the average apparent friction angle was about 24.5 degrees. The maximum depth was measured as approximately 35 m. The landslide mass traveled a distance of about 800 meters, deposited much debris onto National Route 57, and severely damaged a section of the JR Hoho railway track running parallel to the highway (The Japan Time, 2016). This landslide also destroyed an important bridge connecting Minamiaso village to the city of Kumamoto.

3.2 Methodology

3.2.1 Ring-shear Simulator with Simulated Earthquake Waveform for Landslides Triggered by Earthquake

A high-stress, dynamic-loading, undrained ring-shear apparatus ICL-2 (Sassa et al. 2014a, b) was used to test the likely behavior of the brown sample Aso-1 during the M7.3 earthquake. This sample was taken from the area where sliding was assumed to have occurred. Information about the ring-shear apparatus ICL-2 and its testing procedures are available in (Sassa et al. (2014a, 2014b).

The ring-shear test was conducted according to the following procedure:

Firstly, we saturated the sample (taken from the soil layer where the sliding surface was formed) by de-air system one day before testing.

Then, the sample was placed in the shear box, saturated by replacing pore air by CO₂ gas and then replacing CO₂ by de-aired water. The pore pressure parameter ($B_D = \Delta u / \Delta \sigma$) was measured to confirm full saturation ($B_D \geq 0.95$) (Sassa et al. 2010).

The normal stress acting on the sample before and during shearing was set according to the self-weight of the soil layer.

Finally, a cyclic-loading and seismic-loading test using the simulated Kumamoto earthquake waveform were performed under undrained conditions. Shear resistance, pore-water pressure, and vertical displacement of the sample were monitored along with the progress of shear displacement.

3.2.2 Integrated Simulation LS-RAPID for the Initiation and Motion of Landslides Triggered by Earthquake

LS-RAPID is an integrated computer model which can simulate the initiation and motion of landslides triggered by earthquakes. Ring shear test results were input into LS-RAPID as the landslide's dynamic parameters. The M7.3 Kumamoto earthquake wave recorded at KMM005 with 3 components (Figure 3.8) was input to the LS-RAPID model as the triggering seismic parameter of the two landslides.

3.3 Sampling, Testing and Results

3.3.1 Sampling

Two samples were taken from the collapsed area of Takanodai landslide (Figure 3.4), which was mainly composed of strongly weathered lava. Figure 3.6a and Figure 3.6b present the sampling site of Aso-1 and Aso-2 in which the soils change

in color from brown (upper layer, sample Aso-1) to black (lower layer, sample Aso-2). This phenomenon expresses the oxidation state of iron contained in the soil: the lower layer is black in color from iron present in the reduced state (Fe^{2+}) from being submerged in groundwater, while the oxidized part is brown (from Fe^{3+}) (Fukuoka, et al. 2004, Sassa, et al. 2005).

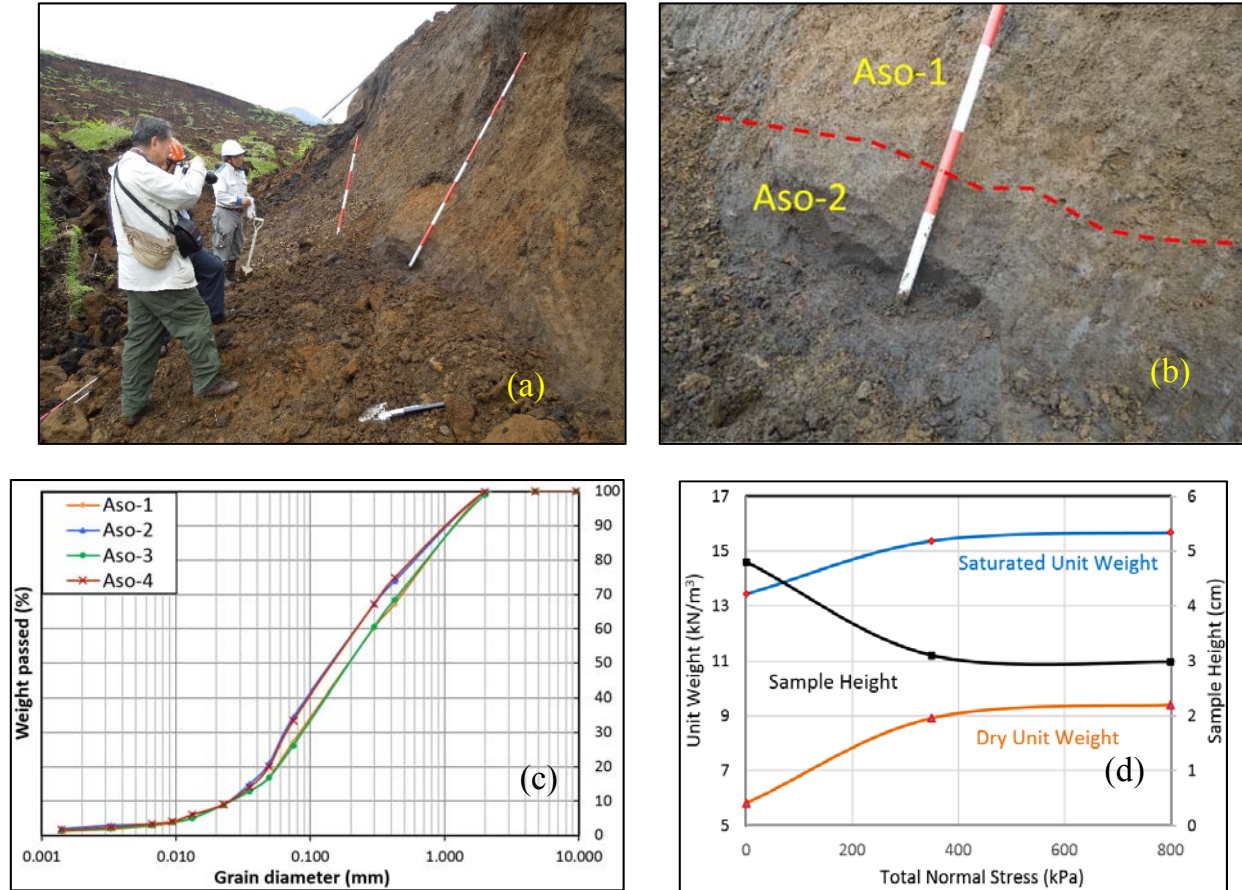


Figure 3.6 Sampling site of Aso-1 and Aso-2 (a, b), grain-size distribution (c) and unit weight (d) of the samples taken from the two landslides

The grain-size distributions of samples Aso-1 and Aso-2 from this landslide are plotted in Figure 3.6c, along with those of samples Aso-3 and Aso-4. The two samples are quite similar although sample Aso-1 contained a slightly higher proportion of sand than Aso-2. According to other basic tests, samples Aso-1 and Aso-2 had specific gravities of 2.66 g/cm^3 and 2.64 g/cm^3 , respectively. To estimate the unit weight of Aso soils, we consolidated sample Aso-1 in the ring-shear apparatus in a saturated condition. The consolidation stress, sample height, dry unit weight and saturated unit weight are shown in Figure 3.6d. The saturated unit weight of the sample was approximately 15.4 kN/m^3 at 350 kPa consolidation stress, and 15.7 kN/m^3 at 800 kPa . The dry unit weight was 8.9 kN/m^3 and 9.4 kN/m^3 ,

respectively. We used a single value of 15 kN/m^3 for the entire area in our numerical simulation.

Grain-size distributions of these two samples Aso-3 and Aso-4 are also presented in Figure 3.6c. They show close similarity (in terms of size-distribution curve) with samples Aso-1, Aso-2 taken from the first landslide area. The brown sample (Aso-3) had a specific gravity of 2.66 g/cm^3 , and black sample (Aso-4) of 2.64 g/cm^3 .

3.3.2 Un-drained Cyclic Loading Test

This test was conducted to examine dynamic behavior of the sample and investigate the seismic acceleration necessary to cause the landslides (Figure 3.7a, b). Initially, the Aso-1 sample was fully saturated ($B_d=0.94$) and consolidated at around 450 kPa normal stress, and 250 kPa shear stress in drained condition (the test was carried out before obtaining the soil unit weight so we tentatively estimated these values of normal stress and shear stress). This initial stress state corresponded to the slope angle of 35° . Then the shear box was changed to the undrained condition by closing the valves and applied with the control signal for the undrained cyclic loading test (Green line in Figure 3.7b).

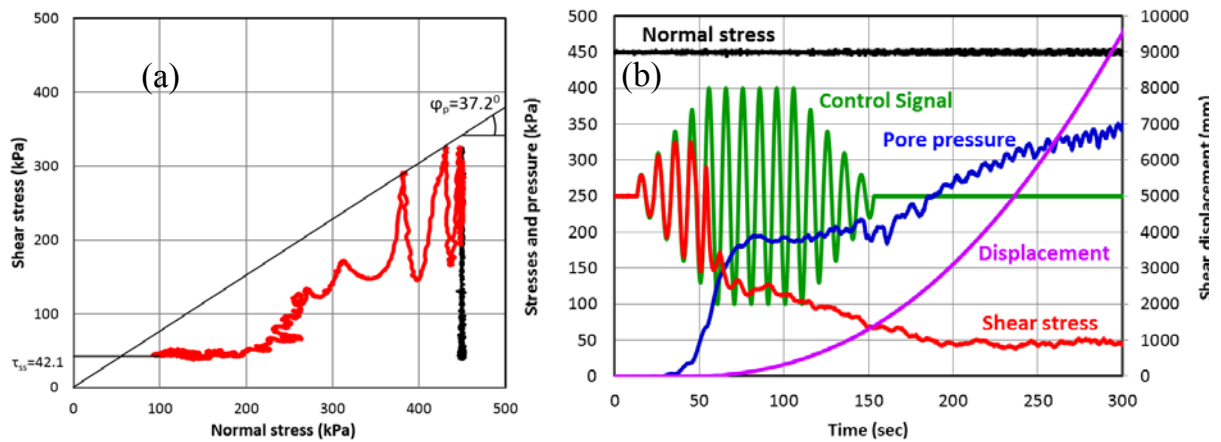


Figure 3.7 Stress path and time series data of undrained cyclic loading test on Aso-1 sample. $B_d=0.94$, frequency of 0.1 Hz, shear stress step of 30 kPa.

During the test, normal stress was kept constant (black line) and shear stress was loaded step by step until 5 cycles with sine wave, in which each step was increased ± 30 kPa. It was expected that the sample would fail before the loading of fifth cycle when the final shear stress reached 400 kPa. After that, five loading cycle were kept constant before the cyclic shear stress was reduced to 250 kPa in 4 steps. The shear resistance, pore-water pressure, and shear displacement are plotted in Figure 3.7b by red line, blue line and purple line, respectively. The shear stress reached the

failure line and decreased after the peak of the third loading cycle due to generation of excess pore water pressure. As shown in the stress path, the peak shear resistance was reached at 325 kPa, friction angle during motion of 37.2° . The necessary seismic acceleration to cause failure was estimated to be 168 cm/s/s. The pore water pressure in this test was not well monitored due to the material being fine. The stress path shows the steady state shear resistance is 42.1 kPa. Shearing stopped after 300 seconds and the shear displacement reached 10 meters.

3.3.3 Un-drained Seismic Loading Test

This test is the most advanced and complicated test of the new ring-shear apparatus to simulate an earthquake-induced landslide (Dang 2015). The earthquake record was obtained from KMM005, an observation station of K-NET (Kyoshin network), which is a nation-wide strong motion seismograph network operated by the National Research Institute for Earth Science and Disaster Resilience (NIED). KMM005 is located at 10 km west of the landslide area and 25 km northeast of the earthquake epicenter. The monitored nearby maximum seismic accelerations are 420 cm/s/s at KMM007, 827 cm/s/s at KMM006, 525 cm/s/s at KMM005, 346 cm/s/s at KMM004. We adopted the monitored seismic record at KMM005 (the nearest monitoring station to two investigated landslides). The distribution of peak ground acceleration of Kumamoto earthquake was introduced by (Furumura 2016).

We used EW component of the earthquake record at KMM005 (Figure 3.8) as the single triggering factor of the landslides and calculated the shear stress acting on the sliding surface based on the section of the Aso-ohashi landslide.

According to the calculation method presented in (Sassa, et al. 2014b), when an earthquake occurs and a seismic acceleration is loaded, the loaded seismic stress acting on the base of a soil layer is expressed by $am=k \cdot mg$. When k is called the seismic coefficient, which is the ratio of the seismic acceleration (a) and gravitational acceleration (g), namely $k=a/g$; mg is the weight of the soil column. The calculated assumed shear stress acting on the sliding surface of the Aso-ohashi landslide during the M 7.3 Kumamoto earthquake is shown in Figure 3.9. As the result of the undrained cyclic-loading test, the additional shear stress required to trigger the landslides is 75 kPa (325-250) corresponding to 168 cm/s/s which is less than the maximum value of the assumed shear stress and the value of monitored acceleration at nearby stations. It was expected that landslides must be occurred during loading of the seismic wave.

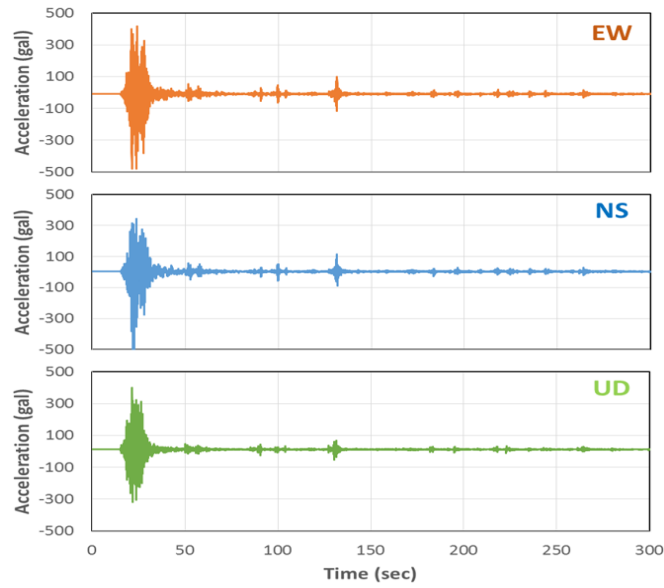


Figure 3.8 Three components of the M 7.3 Kumamoto earthquake recorded at KMM005 site (From K-net of National Research Institute for Earth Science and Disaster Prevention)

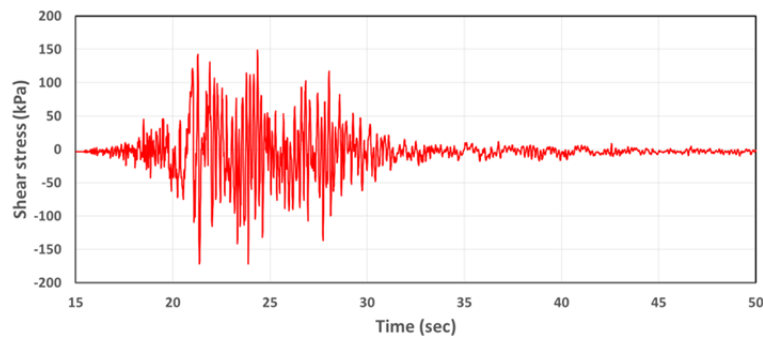


Figure 3.9 Hypothesised shear stress during the M7.3 Kumamoto earthquake acting on the sliding surface of the Aso-ohashi landslide.

Figure 3.10a and Figure 3.10b show the stress paths and time-series data for the undrained seismic loading test. The sample (Aso-1) was initially saturated with B_d value of 0.95 then consolidated to the normal stress of 350 kPa and shear stress of 245 kPa in a drained condition to avoid generation of excess pore-water pressure. These stresses correspond to a slope angle of $\arctan(245/350)=35.0^\circ$ and a sliding mass thickness of 35m with a unit weight of 15 kN/m^3 . After the initial stresses reached these predetermined values, the shear box was changed to the undrained condition by closing all water valves. The EW component of the Kumamoto earthquake wave was loaded as the additional shear stress (with a five-times slower rate to allow pore-water pressure to be accurately monitored). The green line indicates the control signal of seismic loading. The maximum value was 394.2 kPa and the minimum value was 76.2 kPa. The time-series graph (Figure 3.10b) shows a

large decrease of shear stress (red line) and rapid generation of excess pore-water pressure (blue line). Failure occurred at about 20 seconds from the start of the earthquake, with the peak shear resistance of 314.2 kPa. At the same time, pore-water pressure increased rapidly and reached the highest value of 300 kPa, close to the applied normal stress. Due to this pore-water pressure, the effective stress path decreased rapidly along the failure line and reached the steady-state shear resistance. This phenomenon is called “sliding surface liquefaction” (Sassa, et al. 2014b). The calculated peak friction angle was 41.8° , the friction angle during motion was 36.1° , and the steady state shear resistance was 40.0 kPa. The peak shear-resistance value suggested that a smaller earthquake shaking would have been capable of causing failure of the landslides.

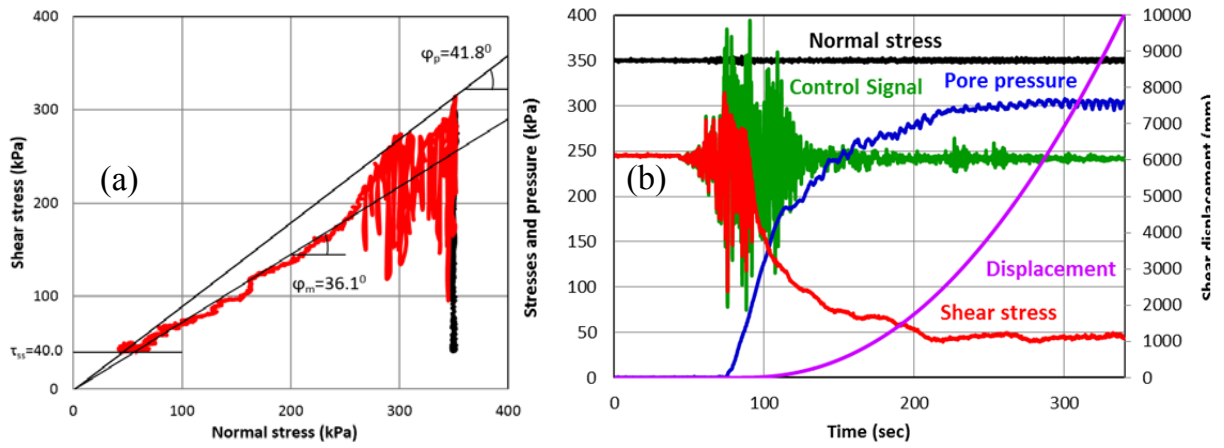


Figure 3.10 Stress path and time series data of undrained seismic loading test on Aso-1 sample using the simulated EW component of Kumamoto earthquake waveform at site KMM005 ($B_d=0.95$)

3.4 Application of the Computer Simulation Model (LS-RAPID) to the Kumamoto Landslides

3.4.1 Landslide Dynamics Parameters

We used the integrated landslide simulation model (LS-RAPID) version 2.1 which can simulate the initiation and motion of landslides triggered by earthquakes or/and rainfall. The basic concept, characteristics, and simulation procedures for this software are detailed in Sassa, et al. 2010 and He, et al. 2014. All parameters of the two landslides (Aso-ohashi landslide and Takanodai landslide) used in the LS-RAPID models are listed in Table 3.1. Most of the soil parameters were obtained from the undrained seismic loading ring-shear test, others were from field investigation, and assumptions based on likely conditions of the landslide areas.

Table 3.1. Properties of Kumamoto soil samples used in LS-RAPID simulation for Takanodai and Aso-ohashi landslides

Parameters used in simulation	Value		Source
	Takanodai landslide	Aso-ohashi	
Shear strength parameters			
Steady state shear resistance (τ_{ss} , kPa)	10	40	Test data
Lateral earth pressure ratio ($k=\sigma_h/\sigma_v$)	0.3-0.7	0.3-0.7	Estimate
Friction angle at peak (φ_p , degree)	41.8	41.8	Test data
Cohesion at peak (c , kPa)	10	30	Estimate
Friction angle during motion (φ_m)	36.1	36.1	Test data
Shear displacement at the start of strength reduction (DL, mm)	4	4	Test data
Shear displacement at the start of steady state (DU, mm)	300	300	Test data
Pore pressure generation rate (B_{ss})	0.7-0.90	0.7-0.90	Estimate
Total unit weight of the mass (γ_t)	15		Test data
Triggering factor			
Excess pore pressure ratio in the fractured zone (r_u)	0.4	0.2	Assumed
The M7.3 Kumamoto earthquake (acceleration, cm/s ²)	Max: 418		Wave form recorded at KMM005
Unit weight of water (γ_w , kN/m ³)	9.8		Normal value
Parameters of the function for non-frictional energy consumption			
Coefficient for non-frictional energy consumption	1.0		See Sassa, et al. 2010
Threshold value of velocity (m/s)	100		A round number greater than possible velocity
Threshold value of soil height (m)	100		A round number greater than possible maximum depth

The first parameter is the steady-state shear resistance (τ_{ss}). We use a value of 40 kPa for the Aso-ohashi landslide based on the undrained seismic-loading test result (Figure 3.10a, b). According to the field survey, Takanodai landslide is a moderate-

shallow landslide with the depth of sliding surface less than 20 m (the Aso-ohashi landslide's depth is 35 m). In addition, the body of the Takainodai landslide was more weathered than that of the Aso-ohashi landslide. Therefore, we assumed $\tau_{ss}=10$ kPa for Takainodai landslide.

Lateral earth pressure ratio k : 0.3 - 0.7. We estimated the ratio to be 0.3 in the stiff slope and 0.7 in the lower part and the flat area.

Friction angle at peak ($\phi_p=41.8^\circ$), friction angle during motion ($\phi_m=36.1^\circ$), shear displacement at the start of strength reduction (DL=4 mm), shear displacement at the start of steady state (DU=300 mm) obtained from the ring-shear test result. DL and DU were obtained from the shear resistance and shear displacement relationship (Figure 3.11).

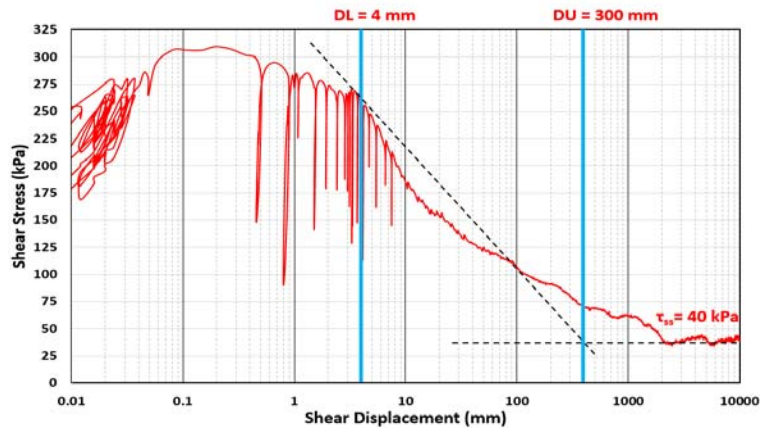


Figure 3.11 Relationship between shear stress and shear displacement of the undrained seismic loading test

Pore-pressure generation rate $B_{ss}=0.7-0.95$ (Takanodai landslide) and $B_{ss}=0.7-0.99$ (Aso-ohashi landslide). The pore-water pressure generation rate is 1.0 in the fully saturated state and 0 in the dry state (Sassa, et al. 2012). At the time of field survey, groundwater was found seeping from the surface of the slope on which the landslides occurred. This suggested that these landslides were probably saturated at the time of failure. So the pore-pressure ratio was assumed to be 0.7-0.90 in the source area of the two landslides and 0.99 along the river under the Aso-ohashi slope (completely saturated).

Total unit weight of the soil mass (γ_t)=15.0 kN/m³ which was estimated from the consolidated sample in the shear box (Figure 3.6d).

As explained in previous part, persistent groundwater was present within the soil layers of these landslides as suggested by the soil color varying from black (due to

reduction) to brown (due to oxidization). In addition, there was groundwater seeping on the Takanodai slope after the landslide had occurred. To simplify the simulation, we assumed a pore-water pressure ratio before earthquake $r_u=0.2$ for Aso-ohashi landslide and $r_u=0.4$ for Takanodai landslide.

The M7.3 Kumamoto earthquake wave recorded at KMM005 with three components (EW, NS, and UD in Figure 3.8) was input to the LS-RAPID model as the triggering seismic parameter of the two landslides.

3.4.2 Simulation Result of the Landslide

0 demonstrates the simulations of Takanodai landslide (left column) and Aso-ohashi landslide (right column). The blue ball zones represent stable parts, red ball zones show the unstable parts when the earthquake occurred.

Simulation started with a pore-water pressure ratio r_u of 0.4 in the case of Takanodai landslide and 0.2 in the case of Aso-ohashi landslide and the earthquake began but no motion occurred.

At 6.5 s, the main shock of the earthquake struck the area and failure occurred in three parts of the Takanodai area and in the main slope of Aso-ohashi landslide.

At 35 s, the earthquake had ceased, all three parts of Takanodai landslide were formed and the Aso-ohashi landslide mass had reached the river valley at the toe of the slope.

At 72.5s, Takanodai landslide mass stopped moving, but Aso-ohashi landslide mass continued moving along the Kurokawa River.

At 156.5s, the Aso-ohashi landslide mass stopped moving.

In comparison with the images taken by UAV (Figure 3.4 and Figure 3.5), the movements of the two landslides in the computer simulations appears to be similar to the real cases. The computer simulation also reproduced the rapid and long traveling motion

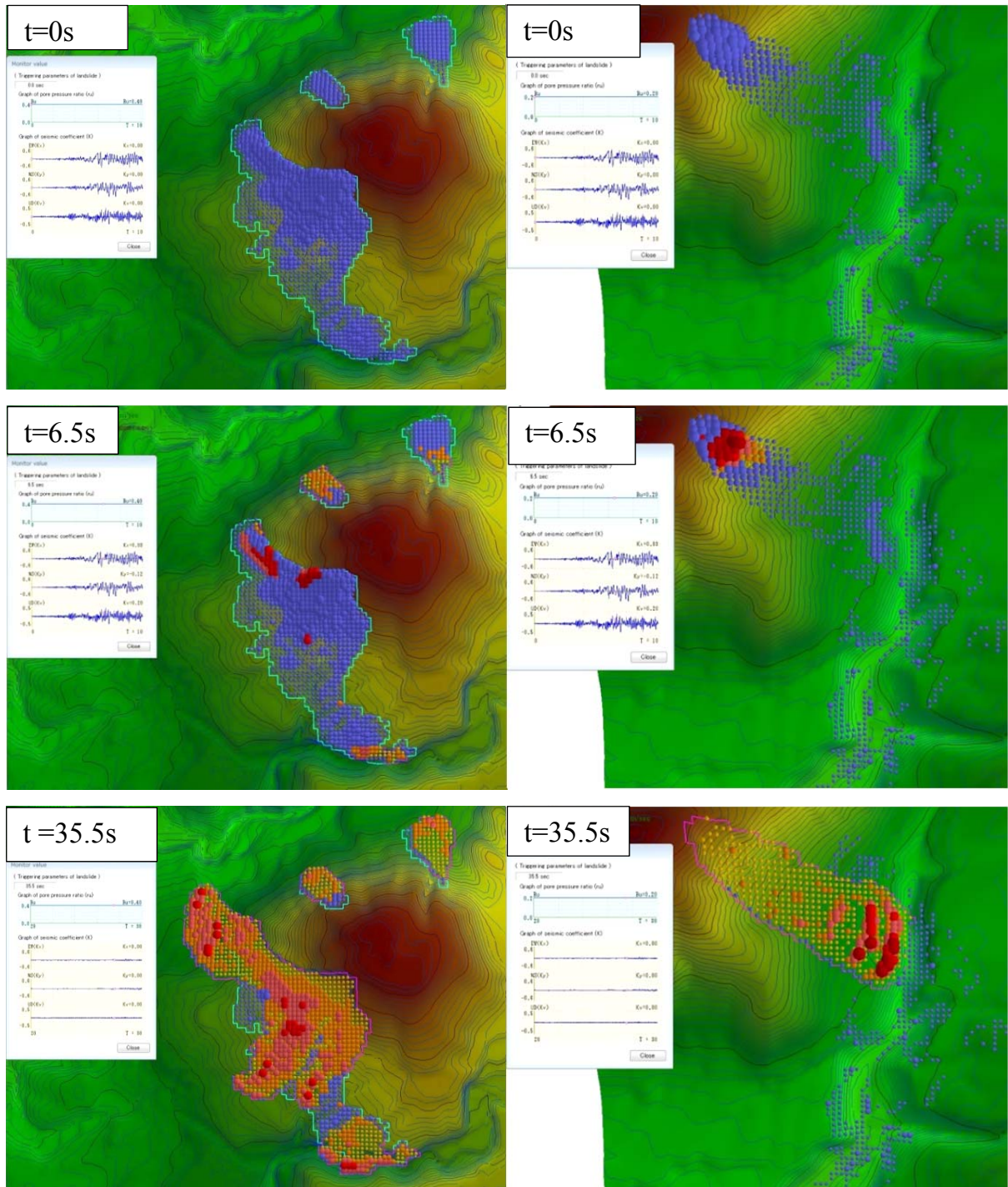


Figure 3.12 Simulation result Takanodai landslide (left side) and Aso-ohashi landslide (right side) due to 16 April 2016 Kumamoto earthquake

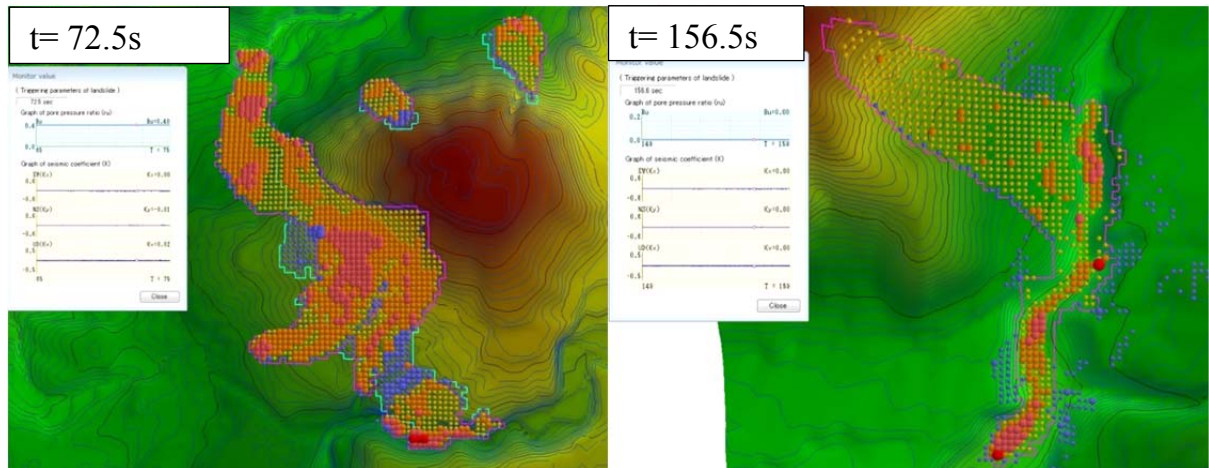


Figure 3.12 (continued)

3.5 Summarization

An undrained, dynamic-loading ring-shear apparatus (ICL-2) was used to study two rapid and long-runout landslides triggered by the largest earthquake of the April 2016 Kumamoto earthquake series in Kumamoto Prefecture. Parameters obtained from field investigation and laboratory experiments were used in a landslide computer simulation (LS-RAPID). Pore-pressure ratio $r_u=0.2$ (for Aso-ohashi landslide) and $r_u=0.4$ (for Takanodai landslide) and the seismic acceleration of the M7.3 Kumamoto earthquake wave recorded at the KMM005 station were used as triggering factors. The computer simulation (LS-RAPID) reproduced the landslides with a similar travel distance and distribution of mass to that shown in the UAV images.

Chapter 4 Hazard Assessment of the Precursor Stage of Landslides

4.1 Introduction

In this Chapter, the main results from testing are presented for one of the project's pilot areas that was believed to be a precursor stage of a landslide at the Haivan Railway Station in central Vietnam. Within the project framework, a movement-monitoring system was installed and core samples were taken from an 80-m deep drill hole at the Haivan Station area. Three samples were taken from sand layers at depths of 21 m (sample S3), 31 m (sample S2) and 50 m (sample S1). In order to investigate the motion of the landslide, another drill-core (sample S4) from 5.6 to 6.0 m depth was collected; it was assumed to represent the soils on the lower slope. The ring-shear apparatus ICL-2 was used to obtain the main soil parameters for these samples.

The chapter also describes numerical modeling of the Haivan Station landslide using LS-RAPID software. LS-RAPID software is an integrated landslide simulation model developed to assess the initiation and motion of landslides triggered by rainfall, strong seismic ground shaking or their combination (Sassa, et al. 2010, 2012). The simulation calculated the critical pore-pressure ratio for landslide occurrence, and the landslide's likely maximum speed, total volume, and depth of landslide debris that could cover the railway. These estimates may serve to raise awareness of the Vietnam national railway sector's vulnerability to landslide impacts.

LS-RAPID software and ring-shear testing have been applied to other landslides (Sassa, et al. 2004a, 2010, 2014a, Dang, et al. 2016, Loi, et al. 2017), reactivated landslides (Arbanas, et al. 2014, Gradiski, et al. 2013) and slopes without any evidence of a landslide (Sassa, et al. 2004b). It is a difficult task to present a landslide risk assessment of a slope which could threaten a very important facility such as a railway station or a cultural heritage. This is the first research to use drill-core samples to measure landslide dynamic parameters for a soil taken from a precursor stage of a developing sliding zone, conduct LS-RAPID simulation and

assess the landslide risk to essential infrastructure.

4.1.1 The precursor stage of Landslides

Natural disasters, such as earthquake, volcanic eruption and landslides are maybe led by some signs or early symptoms known as precursory phenomena. The precursory phenomena have been used to predict and estimate natural disaster occurrence. Precursory phenomena of landslides currently are categorized on the changes in time and mechanisms of disasters (Shinro, et al. 2013) as follows:

1) Precursory landslide phenomena in the pre-active stage (the stage of slip-surface formation): The accumulation of ground deformation is may be caused by geomorphological and geological factors that characterize the precursory landslide phenomena in the pre-active stage. For example, mountain creep by gravity in the period before an actual slip surface has formed or in the period before the slip surface has become continuous. Multiple ridges are also considered to be important indicators of developing landslides (Shinro, et al 2013).

The ground deformation sometimes has taken placed for hundreds of thousands years and does not mostly effect our daily lives. However, the precursory phenomena should be considered significantly if it leads to a rockslide or an earthquake-induced landslide.

2) Precursory landslide phenomena in the incipient motion stage: The phenomena at this stage are identified at the sub-surface as weak movements when underground slip occurs. Surface changes at this stage are not clearly observable. They have used gauges measuring cumulative changes or recorded and analyzed underground fracturing and cracking sounds of rocks for underground landslide movement (Fukuda, et al. 2002).

3) Precursory phenomena of the land-moving stage: These are precursory phenomena just before various landslides are recognized on the surface. There are many changes identified as the precursory phenomena are seen. They are cracks on the land surface, small-scale collapse and artesian water spouting from the slopes, turbidity of spring, mountain stream or well water, cracks in buildings, deformed doorways, cracks in walls, inclined walls, trees and telephone pole, changes in water level of ponds and swamps, uplift or subsidence of ground and roads (Shinro, et al. 2013). Deformation of the moving mass and ground water pressure variation cause these precursory phenomena.

Some other changes are often seen in the field that can indicate a potential occurrence of a landslide. The weather conditions, the slope of the slipping ground and the site geology are factors effect these changes. It should be taken into account when these changes are noticed.

4) Precursory landslide phenomena at the landslide-occurrence stage: Sometimes the landslide occurrence and the land itself moving are considered as precursory phenomena when the landslide locations far from residential areas.

Many field surveys have been conducted to identify the precursory phenomena of landslides and estimate the possible scale and depth of slid surface and the future landslide-forming process. However, there are few research reports on the precursory phenomena of landslides. Observing facts and changes before and during landslide occurrence is difficult. Normally, the precursory phenomena are recognized after the landslide occurs.

4.1.2 Study Area-A Precursor Stage Landslide Threatening Haivan Railway Station, Vietnam

Figure 4.1 presents the location and topography of the landslide study site. The Haivan Station (Figure 4.1c) is located on the seaward side of Haivan Pass, which is the provincial boundary between Da Nang city and Hue province. It is an important station along the national railway, and it is estimated that over 30 trains pass this station each day on the only railway from Hanoi to Ho Chi Minh.

In the rainy season, landslides are a serious potential danger to passenger safety. In 1999, a historic storm caused numerous landslides and debris flows around Haivan Pass. These landslides varied in volume from 100 m³ to 10,000 m³ and the Haivan Station was damaged by the largest one (Tien P, et al. 2015, Tam 2005). At the end of 2004, the area around Haivan Station was strengthened with a concrete frame and retaining wall. However, more than 7,000 m³ of rock and soil moved down and covered the railway after the 2005 rainy season. The last landslide occurred in November 2007 (Figure 4.1b). The economic loss was estimated to be about US \$3.4 million. The landslides around Haivan Station in 1999, 2005, and 2007 were relatively small-scale ones, assessed as occurring in the toe of a precursor stage of a larger scale sliding block.

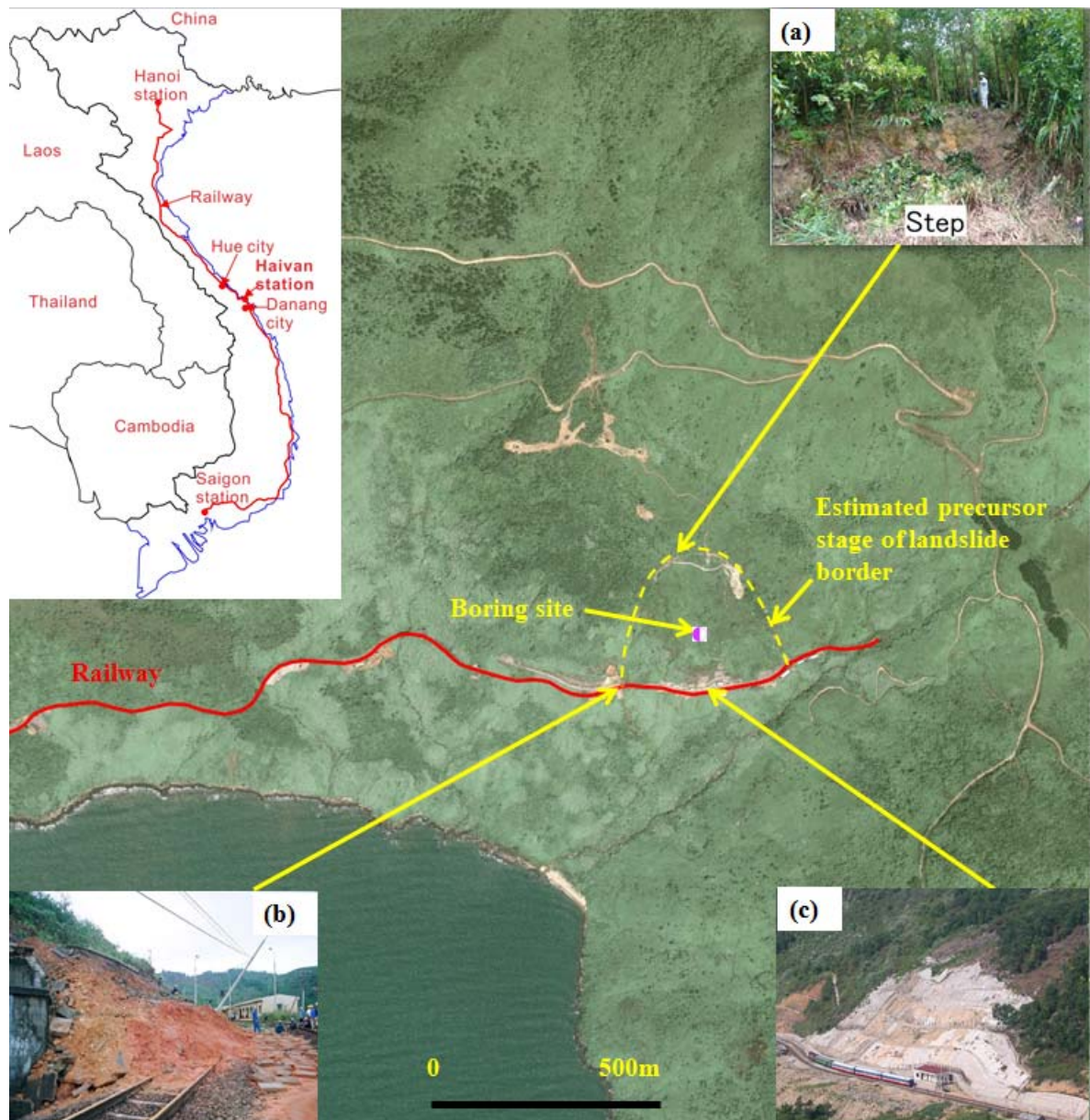


Figure 4.1 Location, topography, geological structure, an estimated precursor stage of deep-seated landslide border on the back-side slope of the Haivan station, in Vietnam. a) The small step where a head scarp of a precursor landslide may be formed; b) A previous landslide in 2007; c) A previous landslide behind the Haivan station

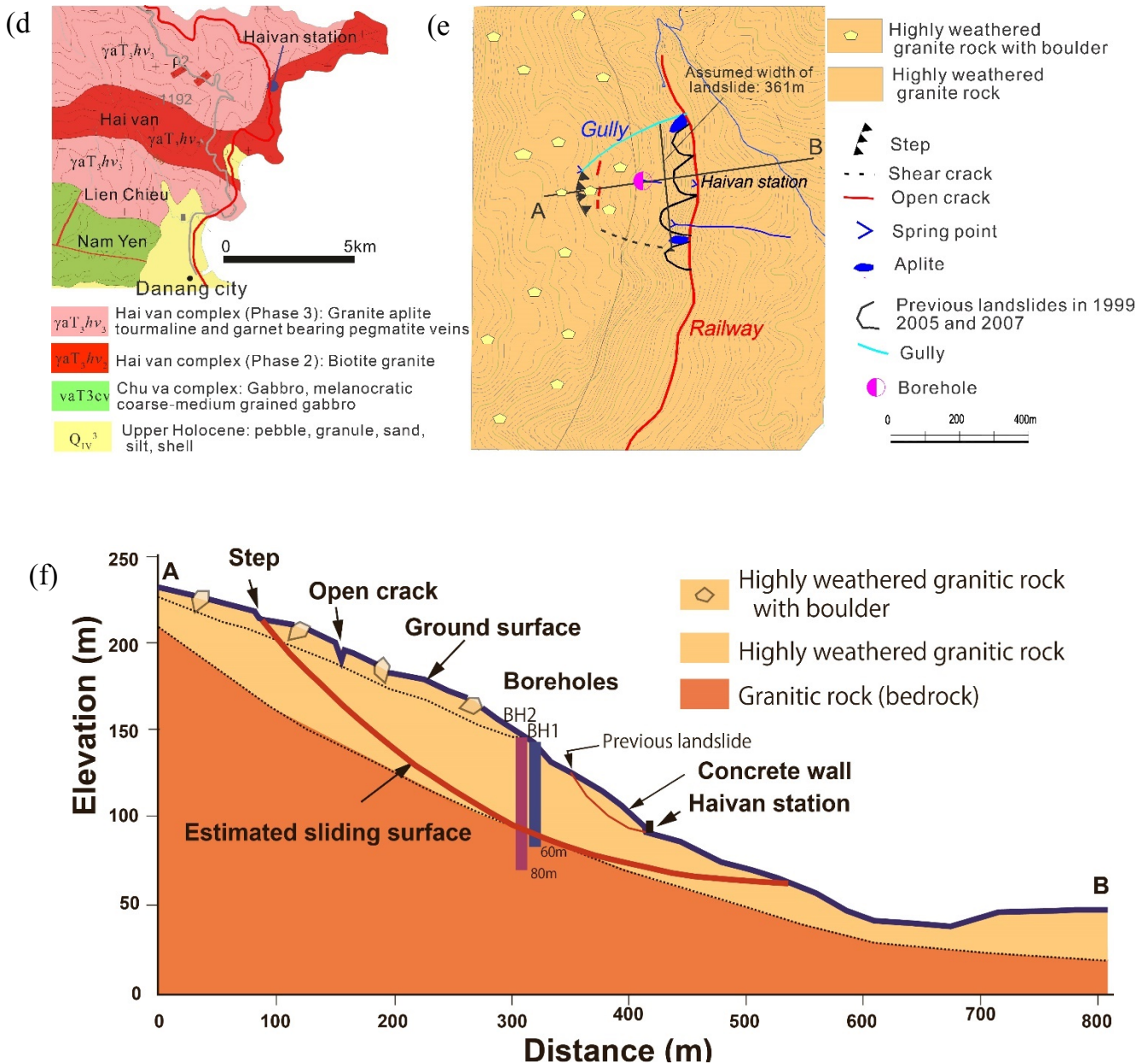


Figure 4.1 (continued) d) Geological map of wider area e) Simplified engineering geological map f) Geological cross section A-B.

Due to the terrain making access difficult, very few geological and engineering geological studies have been carried out in Haivan pass area. A survey was conducted of high-grade metamorphic terranes in the central of Vietnam (Nakano, et al. 2013). At Haivan Pass, sedimentary rocks have been intruded by a cordierite-bearing granitoid which is composed of granite-granodiorite-tonalite and includes several high-grade metamorphic blocks. These blocks display typical migmatitic structures; they are a few centimeters to ten meters in diameter, are commonly rounded or vermicular, lacking sharp edges. The studied samples showed that granitic and gneissic blocks are frequently found to separate clearly, but sometimes

the granite blocks sink into gneissic blocks. The granitoid is unreformed and commonly composed of sub- to euhedral cordierite, biotite, plagioclase, and K-feldspar with relatively coarse-grained anhedral quartz. Gneissose blocks have strong metamorphic fabric that is composed of prismatic or fibrolitic sillimanite, biotite, and elongated cordierite (Nakano, et al. 2013).

Based on the national geological map by the General Department of Geology and Minerals of Vietnam (Figure 4.1d) and lithological description (Figure 4.2), the underlying geology of the area is mainly granite of the Haivan complex, intruded some 200 – 250 million years ago. The Haivan complex includes a number of intrusive phases but only two are present here: Phase 2 ($\gamma\alpha T_3 \text{ } hv_2$) is biotite granite; and Phase 3 ($\gamma\alpha T_3 \text{ } hv_3$) consists of granite aplite, tourmaline- and garnet-bearing pegmatite veins. Deep weathering is primarily restricted to the lower valley walls and floors, gullies and moderate slopes, and appears to diminish in magnitude to the west. In general, the 300-m-high mountain slopes are composed of relatively fresh granite, and granitic talus consisting of large boulders and blocks, resting on thin, poorly consolidated and immature soils containing abundant granitic detritus. Below the 300-m elevation, the weathering profile is thicker and large interfluvial talus deposits occur. Fresh rock is close to the surface at most localities, and crops out commonly as hillside scarps, particularly at higher elevations, while greater weathering is predominantly confined to areas lower on the valley walls. Where the rock possesses moderate to strong foliation, weathering appears to be more extensive. Talus deposits are typical along the slopes of the mountain range. There are two main types of talus slopes: (1) large boulders and blocks that have fallen from the steep slopes or been carried down with debris flows to the valley floor, and (2) secondary deposits on gentle slopes consisting of finer material that has been washed out from the main talus slope by surface and groundwater runoff. The larger talus deposits may be subject to massive water discharge from ground- and surface-water runoff, during or shortly after heavy rain.

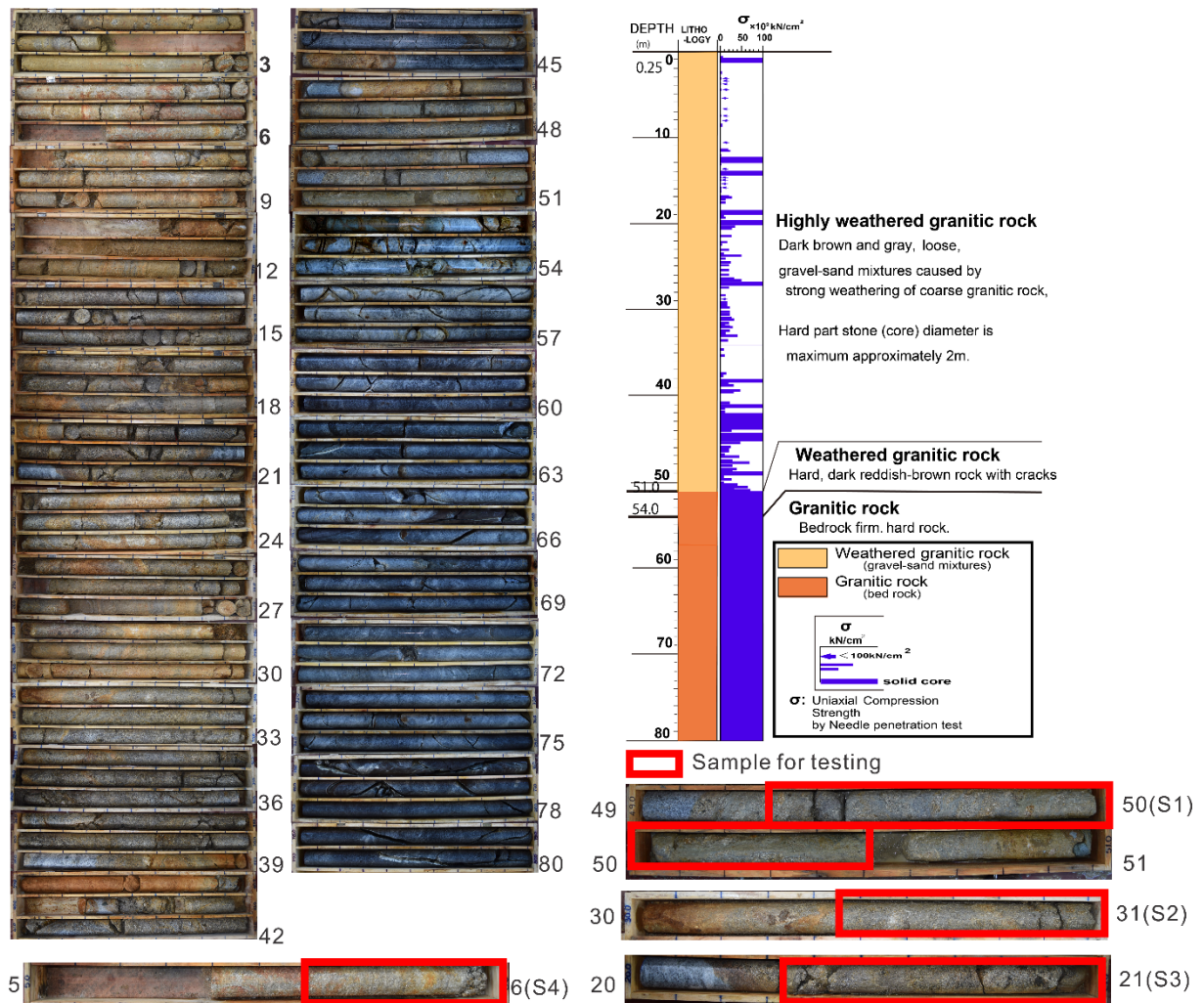


Figure 4.2 Lithological description of the Haivan station borehole and the sampled sections (red outlines: S1, S2, S3 and S4) from weathered layers of the drilling core

Some field investigations were conducted following a centre survey line and some subsidiary survey lines (Figure 4.3) to explore the landslide scarp and body. In this area, hard boulders with several-meter radius are left in the sand by deep weathering granite; and they are scattered on coastal slope like core stones. Therefore, a peculiar landscape has developed. Landform deformation is a factor related to landslide. The WG2 working group found some steps, open cracks and a series of shear cracks as well as a developed gully. The steps (Figure 4.1a and Figure 4.1e) are from 0.5 to 3.0 m in height and may be formed by slope movement. The open cracks (Figure 4.1e and Figure 4.1f) sub-parallel to contour lines were considered to represent an unstable state attributable to a deep-seated slide at the depth of 50 m. A series of shearing cracks (Figure 4.1e) was located at the landslide border. Movement has also deformed the top drainage channel and gabion near the railway track. Some spring points were found in the landslide zone. The water runoff

through the landslide body has caused surficial erosion. Some erosion gullies and channels were investigated that have a V-shaped cross section. The largest gully (Figure 4.1e) was developed on the left-side of the Haivan Station. The eroded gully width ranges from 1.5 m at the slope foot to 5.0 m at the top of mountain. The gully depth varies from 0.5 m to 2.0 m along longitudinal direction. During the rainy season, a large volume of runoff flows through this gully from the top of mountain down to the slope foot and the lower plain on the seaside. The gullies contribute to significant erosion of the hill side and slope foot and so decrease the slope stability. The plan of the estimated precursor stage of landslide was estimated as shown in Figure 4.1e with the illustration of the steps, open cracks, shear cracks as well as previous small landslides around its border.

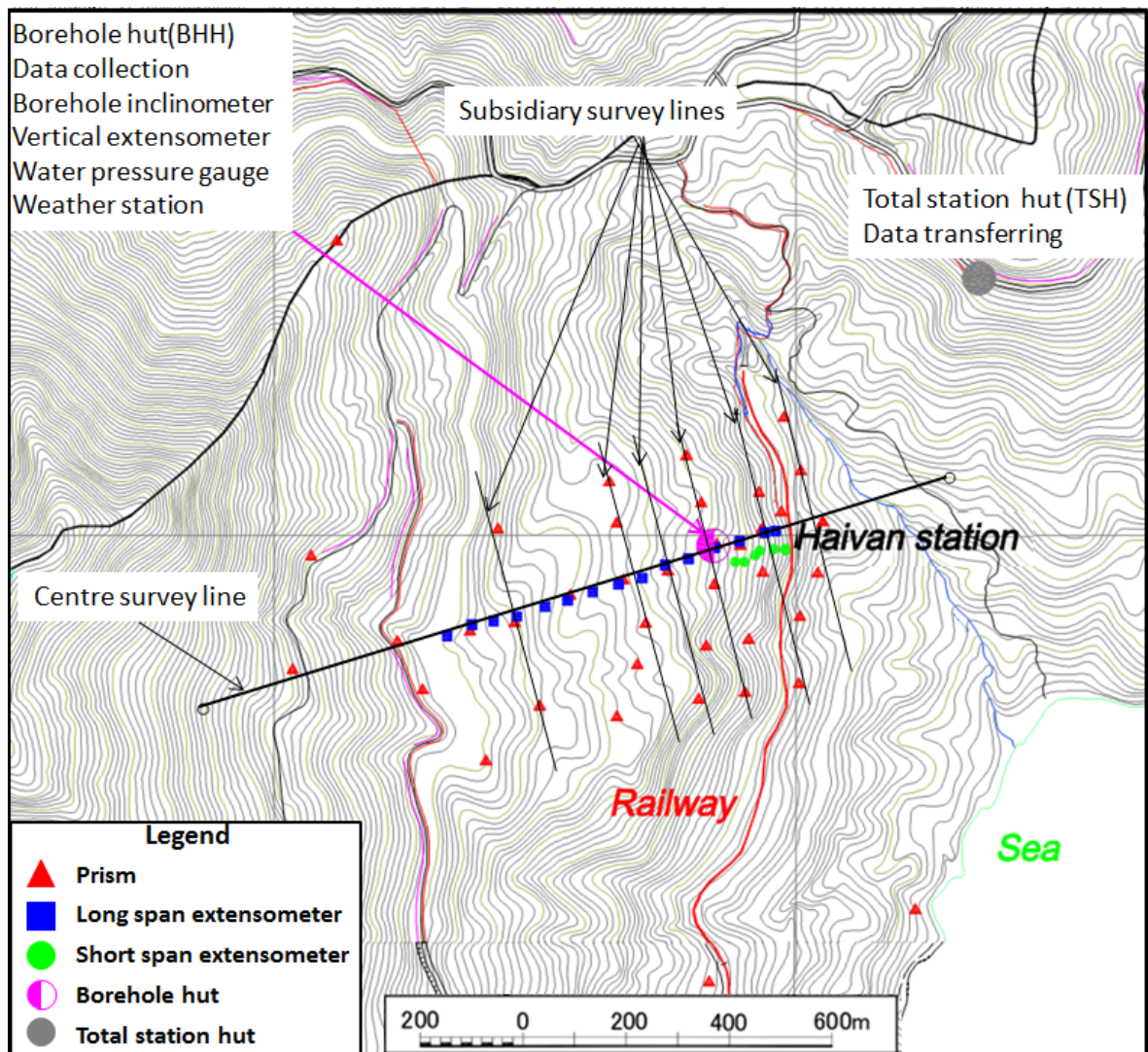


Figure 4.3 Location map of monitoring sensors on the Haivan station slope

The depth of the precursor landslide was estimated from the geological drilling and inclinometer monitoring. The boundary of the granitic rocks is very clear in the

drilling cores (Figure 4.2). The granitic rock is highly to completely weathered to some considerable depth beneath the surface. The vertical geological section at the borehole was described as: depth 0 to 51.0 m, highly weathered coarse granite with clay sand, sand and solid granite; depth 51.0 to 54.0 m, weathered fine granite, hard, dark reddish-brown rock with cracks; below a depth of 54 m was described as granite bedrock. Samples for ring-shear testing were taken from three layers at 20.4-21.0 m, 30.5-31.0 m and 49.15-51.0 m to measure landslide dynamics parameters.

A central cross section was drawn by estimating a circle passing the feature labelled as the “Step” (as the head scarp) and the boundary between highly weathered granitic rock and the granitic rock at Borehole (BH2) as illustrated in Figure 4.1f. The slope consisted of three material layers: highly weathered granite rock with boulders, highly weathered granite rock; and bedrock. The highly weathered granite rock could be fractured easily by finger pressure of 1–50 kg/cm², as indicated by needle penetration tests.

4.2 Monitoring System

4.2.1 Borehole Inclinometer

The Haivan Station landslide is one of the pilot study areas covered by the Vietnam-Japan joint research project. A landside monitoring system has been established in the Haivan Station area. The monitored items are surface displacement, underground displacement, weather conditions and pore-water pressure. The locations of the monitoring sensors are presented in Figure 4.3 and the monitored items are listed in Table 4.1. Major monitoring results used in the analysis for this research are written below. None of the GNSS, long-span extensometer, and robotic total station measurements extended beyond measurement error and so, these are not described in this chapter.

Table 4.1. Monitoring sensors for Haivan station

Monitoring items	Sensors	Amount
Surface displacement	Total station	40 (3 standard points include)
	GNSS	3 (1 standard point include)
	Long-span extensometer	14
	Normal distance extensometer	5
Underground displacement	Borehole inclinometer	9
	Borehole extensometer	2
Weather	Weather station (Rain gauge, Air temperature, Air pressure)	1
Pore-water pressure	Water pressure gauge	2

Inclinometer casing was installed in the 80-m deep borehole (Figure 4.5), and an inclinometer was used to measure the displacement at various depths. The incremental displacement profile was measured twice in a month period. Figure 4.4 shows the inclinometer data from 15 September 2015 to 15 March 2016. It indicated that from 20 to 70 m depth, the displacement at around 50 m depth was not only the largest, but was growing incrementally. The inclinometer displacement clearly showed that formation of a sliding surface was in progress.

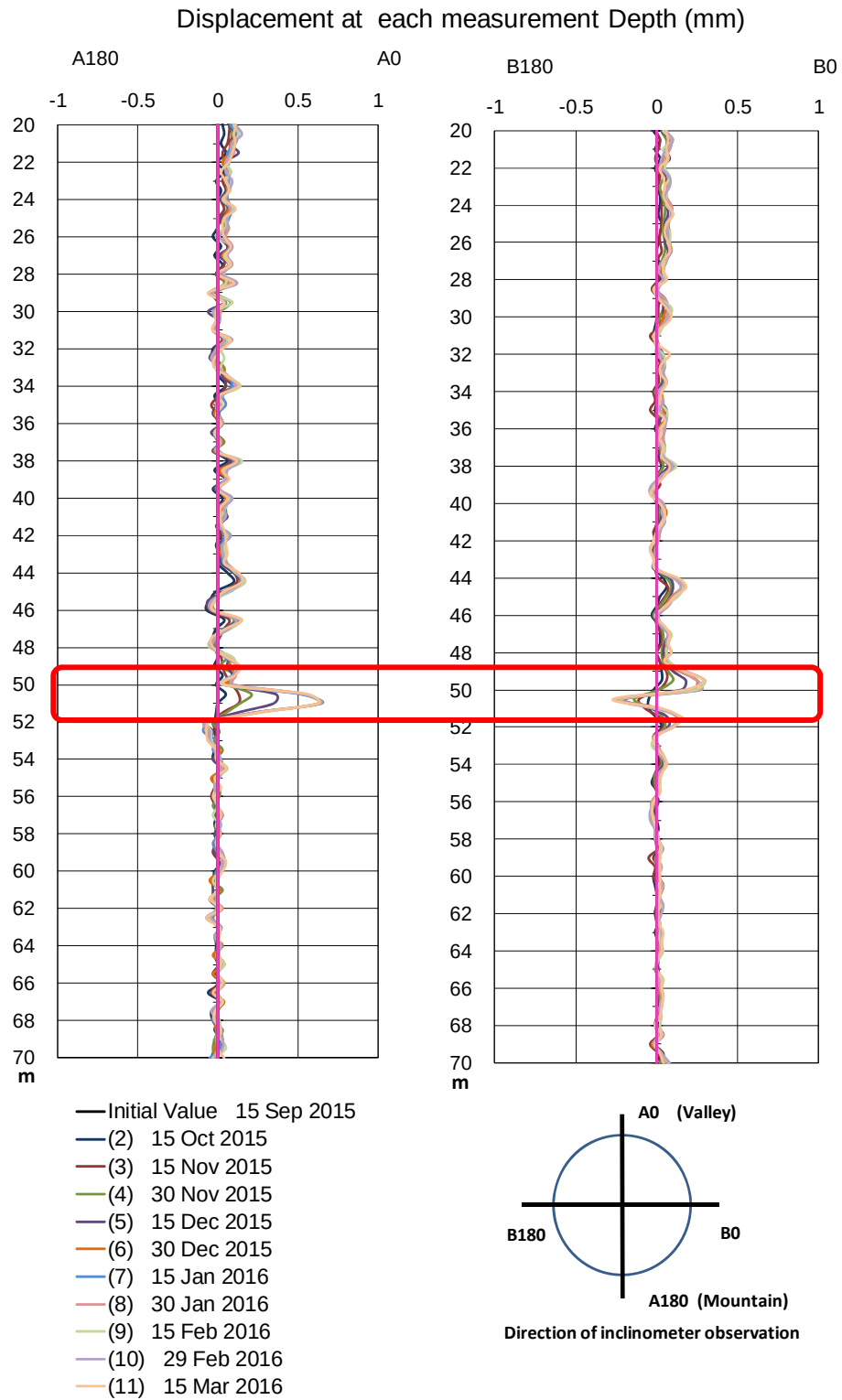


Figure 4.4 Inclinometer measurements at each depth from 15 Sep. 2015 to 15 Mar. 2016

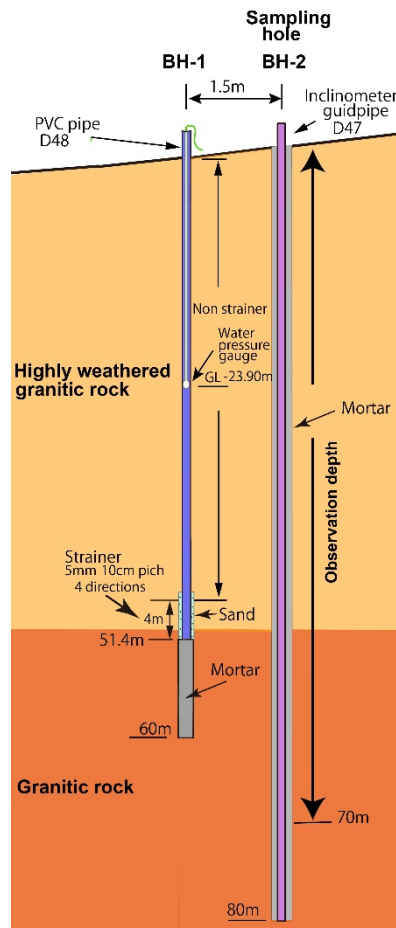


Figure 4.5 The section of BH-1 for pore pressure monitoring and BH2 for inclinometer monitoring

4.2.2 Rainfall and pore-water pressure measurements

Rainfall and pore-water pressure are two important factors that contribute to the formation and development of a landslide. Rainfall data has been gathered by a 0.5 mm tipping-bucket rain gauge installed at the top of the borehole hut since December 2014. Pore-water pressure has been monitored continuously since April 2015 by piezometer in the 60-m deep borehole (BH1). BH1 was drilled to investigate ground water and is 1.5 m distant from the 80-m deep borehole (BH2) which was all core drilling for geological investigation and inclinometer monitoring. Geological inspection of the core samples (Figure 4.2) and inclinometer monitoring (Figure 4.4) suggested that the potential sliding surface was located in the base of the highly weathered granitic rock (0 – around 50 m). The Casagrande standpipe piezometric monitoring method was adopted (Casagrande 1949). Figure 4.5 presents the section of BH-1 for pore-pressure monitoring and BH-2 for inclinometer monitoring. BH-1 is a standpipe and its water inlet is a 4-m long pipe with strainers from 47.4 m to 51.4 m. The outside space between the borehole pipe

and the ground layer was back-filled with sand. The pore pressure at the strainer depth is calculated as the product of (the height of the water table above the measured depth) x (the unit weight of water). This pressure was monitored by a water-pressure sensor hanging in the water at a depth of 23.9 m. The output of this pore pressure sensor converted to show the depth of water table in BH-1. The monitored depth of the water table in BH-1 from 22 April 2015 to 29 June 2016 is presented in Figure 4.6. The highest water table was 15.2 m on 22 December 2015 and the lowest water table was 19.2 m on 29 June 2016. The water-pressure sensor was initially installed at 18.9 m and so the period when the water table was lower than this level (18.9 m) is shown as “lack of observation”. The water pressure sensor was lowered to 23.9 m on 04 December 2015. Thereafter, the depth of water table in BH-1 was monitored until the present.

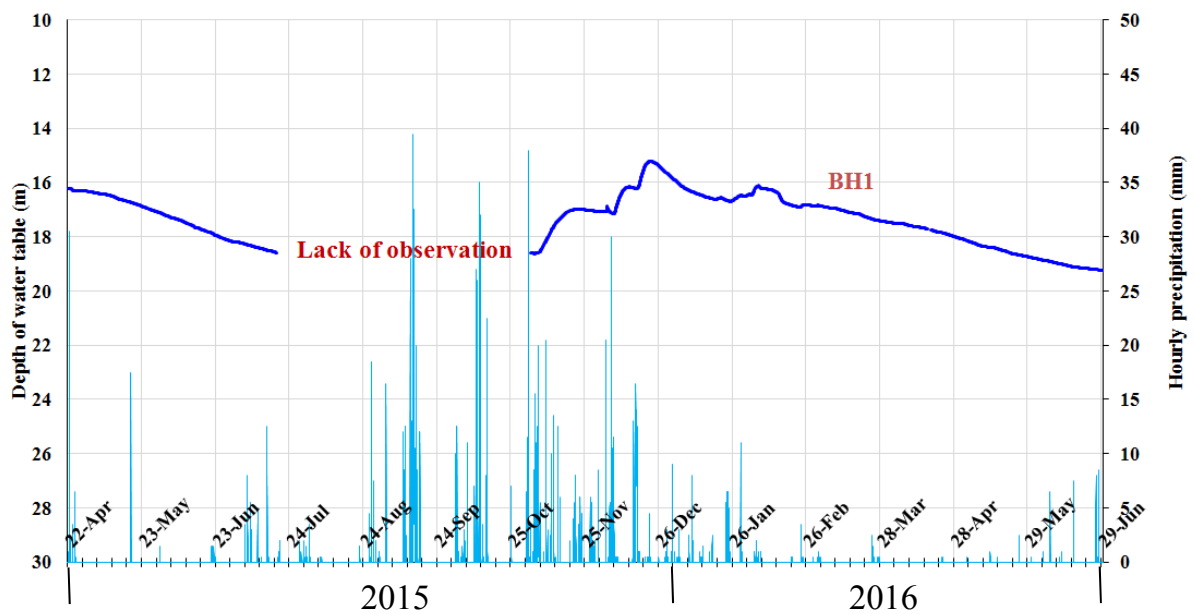


Figure 4.6 Hourly precipitation and water level in BH1 is formed by pore pressure acting in the zone of 47.4-51.4 m from 22 April 2015 to 29 June 2016

4.3 Sampling, Testing and Results

4.3.1 Sampling

The ring-shear apparatus is widely used in slope-stability analysis, because it can simulate a landslide with unlimited shear displacement (Bishop, et al. 1971, Sassa ,et al. 2004a). Since 1984, Sassa and colleagues have developed a series of ring-shear apparatus. The latest version of these devices is a high-stress dynamic-loading undrained ring-shear apparatus, ICL-2 (Sassa, et al. 2014a, 2014b). The design allows the simulation of large, deep-seated landslides. The volume of the

shear box is 300 cm^3 , which is smaller than in the previous series of ring-shear apparatus (Sassa, et al. 2004a). Therefore, ICL-2 is also suitable for laboratory testing when drill cores provide limited samples. For the Haivan core samples, the fully saturated samples were tested following the procedure detailed in Sassa, et al. (2014a).

Basic tests were carried out to obtain mechanical parameters of three drill-core samples. From the grain-size distributions (Figure 4.7), the samples were considered to be well graded soils. They were consolidated in the ring-shear apparatus under different normal stresses from 0 to 1 MPa in a saturated condition. The saturated unit weight of the samples was 21.2 kN/m^3 and the dry unit weight was 18.8 kN/m^3 at 1 MPa normal stress.

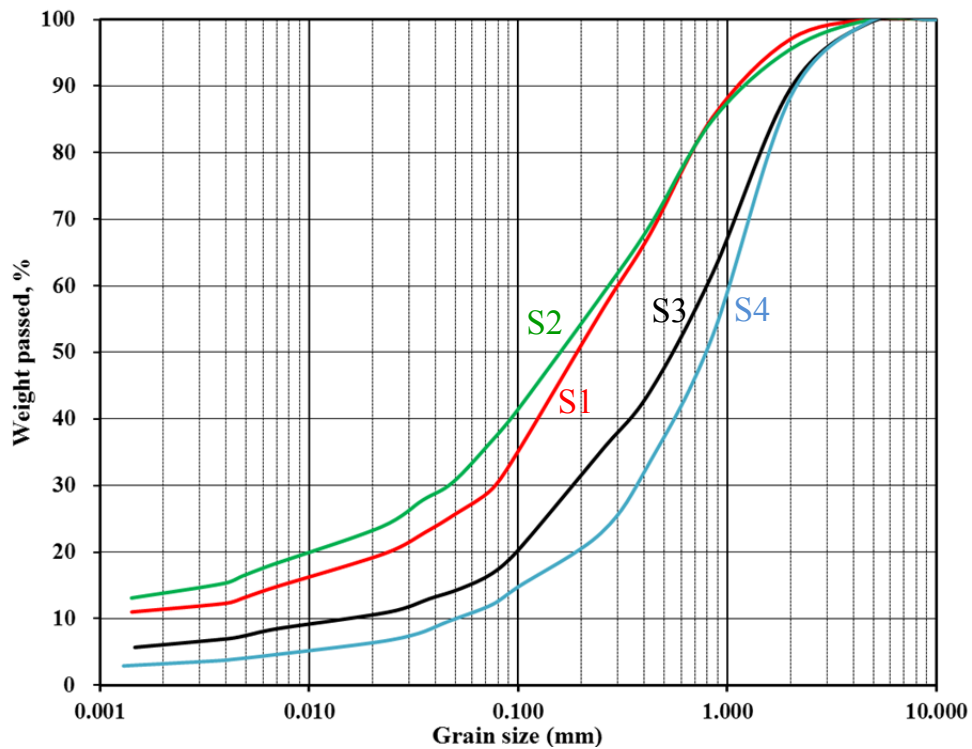


Figure 4.7 Grain-size distribution of Haivan drillcore samples taken from depths of 50 m (S1), 31 m (S2), 21 m (S3) and 6 m (S4)

4.3.2 Drain, shear-speed Controlled

Regardless of the influence of pore-water pressure, a drained test is considered one of the best methods to measure friction angle (Sassa, et al. 2014a). The experiments were performed at a sample saturation $B_d=0.96 - 0.99$. At normal stresses close to 780, 520, and 350 kPa, corresponding to landslide depths of 50, 31 and 21 m (Figure 4.8), the samples were sheared at 0.1 cm/sec under drained conditions. Once a peak shear resistance had been reached, the normal stress was decreased to 0 kPa

at rate of $\Delta\sigma=1$ kPa/sec. The maximum friction angles obtained for the three samples S1, S2, and S3 were 39.0° , 38.7° and 39.8° , respectively. The friction angles during motion for the same samples were 37.7° , 36.8° and 36.6° respectively.

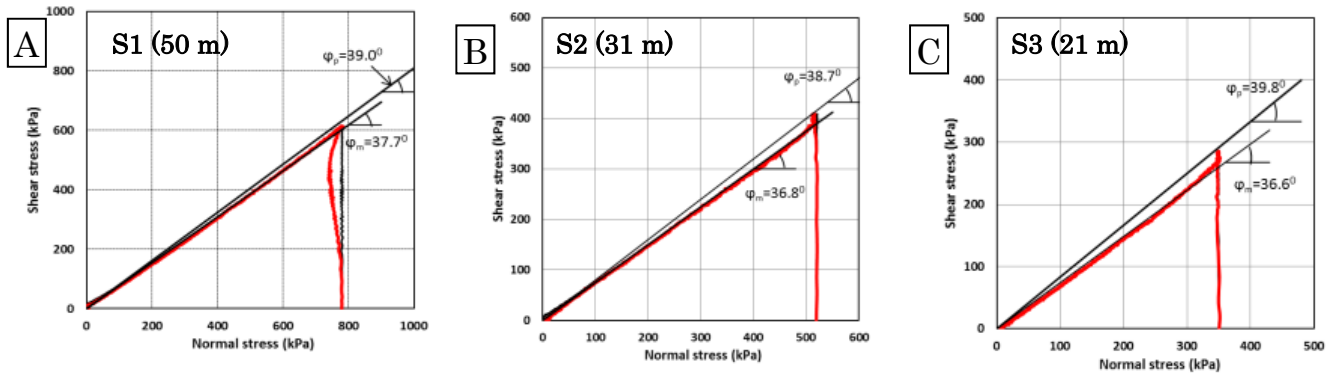


Figure 4.8 Stress paths of drained shear-speed control tests on the three drill-core samples.

(A) Sample S1; (B) Sample S2; (C) Sample S3, $B_d=0.96-0.99$

4.3.3 Undrained, shear-stress Controlled test

Normal stresses of 780 kPa, 520 kPa, and 350 kPa were respectively applied to consolidate the samples. The tests were then conducted under undrained conditions. The pore-water pressure was increased while the shear stress loading was gradually raised at a rate of $\Delta\tau = 1$ kPa/sec until the effective stress path reached the failure line. Shear loading was applied until a steady state was obtained. In Figure 4.9, the red line is the effective stress path and the black line is the total stress path. From the graphs, the values of the peak friction angle are 39.8° (S1), 39.7° (S2), 38.7° (S3). The three peak friction angles are almost the same value. The steady-state shear resistances were 116 kPa (S1), 240 kPa (S2) and 56 kPa (S3), where S2 had the highest value. The mobilized friction angles were 38.0° (S1), 35.3° (S2), and 38.7° (S3).

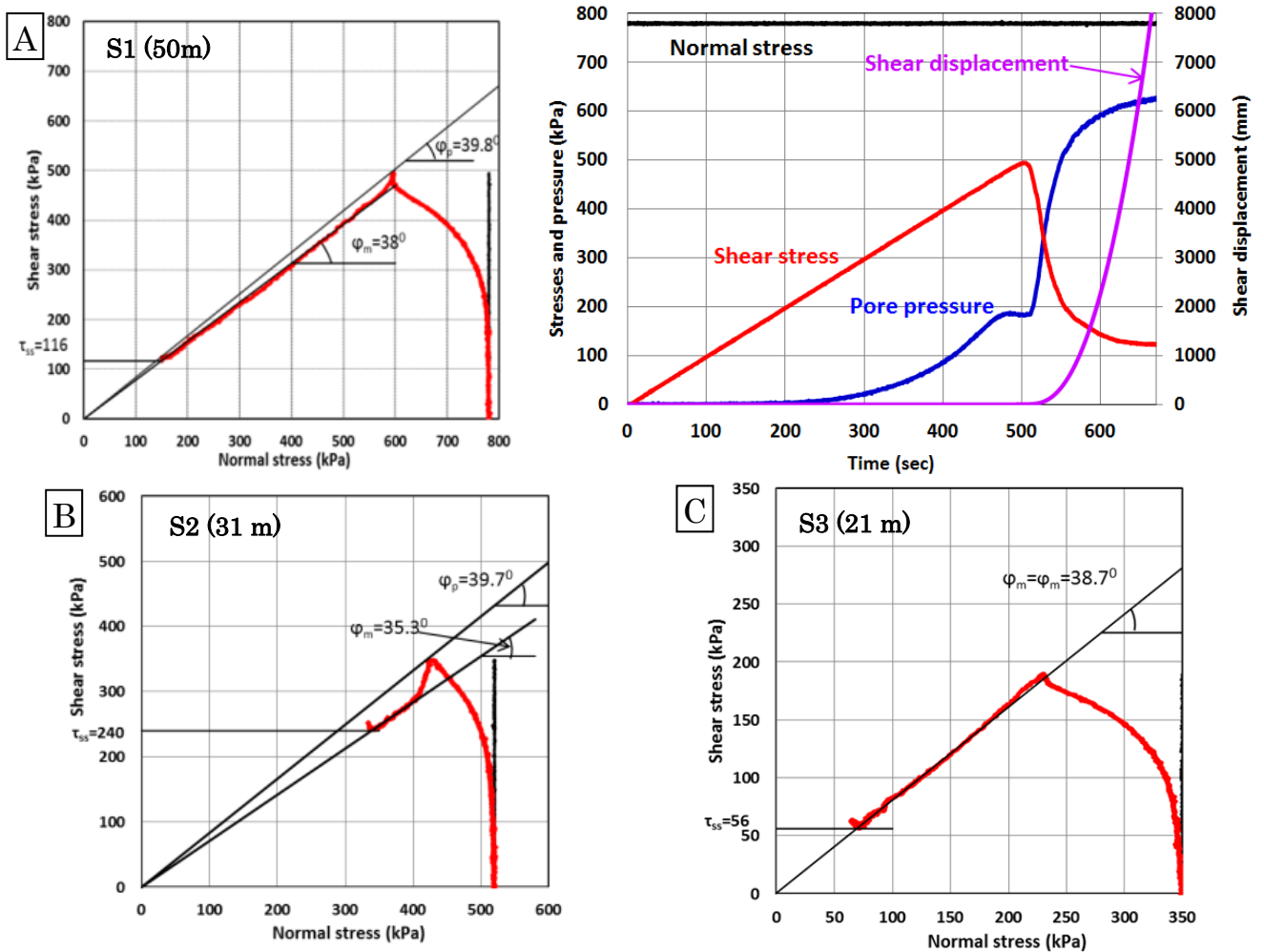


Figure 4.9 Undrained shear-stress control tests on Haivan drill-core samples with $\Delta\tau/s=1$ kPa/s. (A) Sample S1, Normal stress=780 kPa, $B_d=0.96$; (B) Sample S2, Normal stress=520 kPa, $B_d=0.95$; (C) Sample S3, Normal stress=350 kPa, $B_d=0.96$

4.3.4 Pore-water pressure controlled test

Pore-water pressure controlled tests are the most appropriate experiments to simulate the effect of rainfall infiltration and pore-water pressure increase on landslide activation. S1, S2, and S3 samples were processed step-by-step from saturation ($B_d=0.95 - 0.99$), through consolidation to normal stresses of 780, 520, and 350 kPa and shear stresses of 360, 200 and 150 kPa respectively. The initial stress state corresponded to slopes of 24.8° , 21.0° and 23° . These were approximately the actual slope of the landslide blocks (about 25°). To simulate the effect of pore-water pressure in causing the landslide phenomenon, the pore-water pressure was increased at a rate of 1kPa/sec to obtain friction angles of 39.5° , 39.8° , and 38.9° (Figure 4.10). Failure occurred when the pore-water pressure reached 360, 297, and 175 kPa, namely, a pore-water pressure ratio $r_u=360/780=0.46$, $r_u=297/520=0.57$, and $r_u=175/350=0.50$ for samples S1, S2, and S3 respectively.

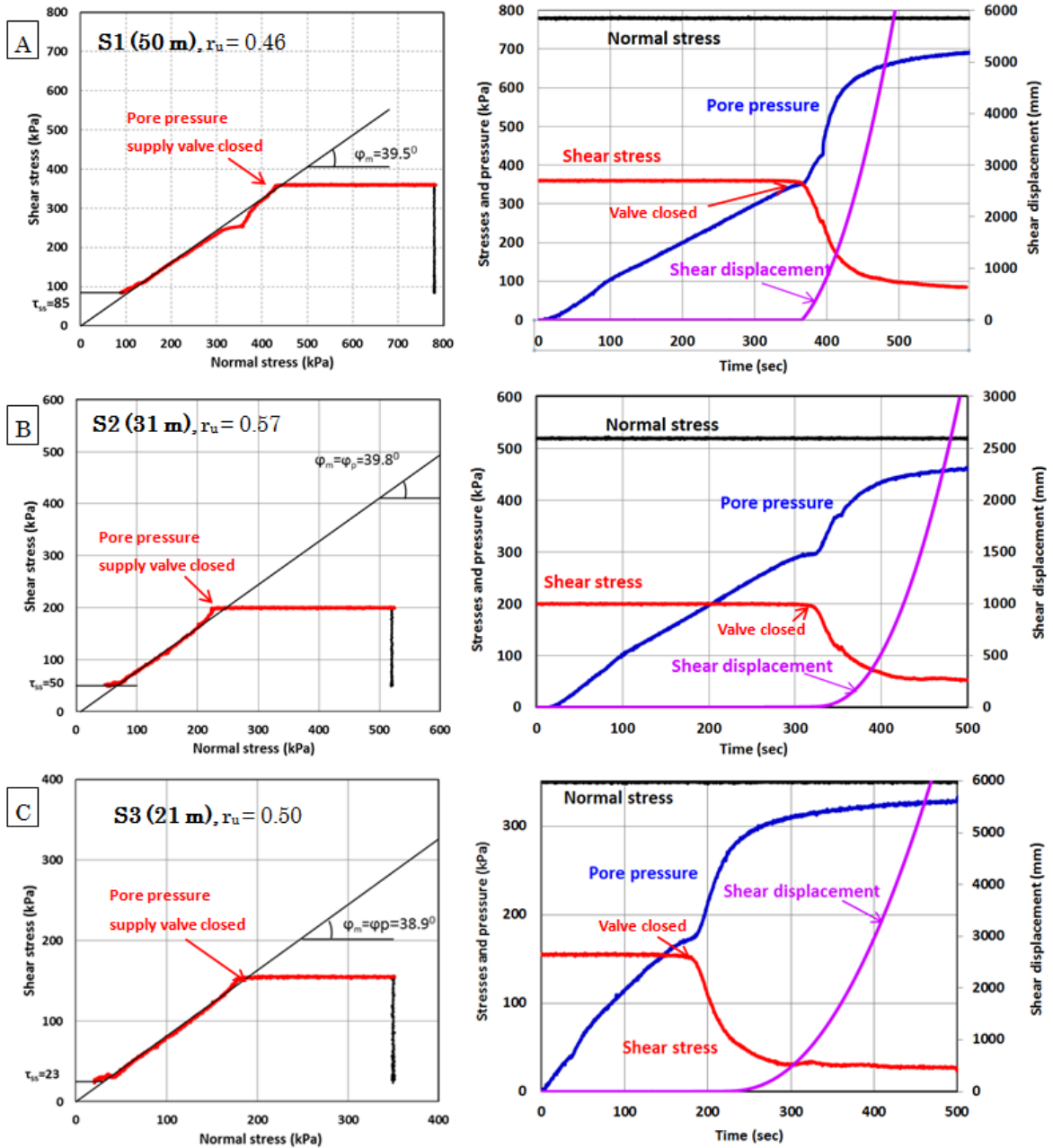


Figure 4.10 Pore-pressure control tests on Haivan drill-core samples with $\Delta u/s = 1$ kPa/s. (A) Sample S1, Normal stress=780 kPa, Shear stress=360 kPa, $B_d = 0.99$; (B) Sample S2, Normal stress=520 kPa, Shear stress=200 kPa, $B_d = 0.96$; (C) Sample S3, Normal stress=350 kPa, Shear stress=150 kPa, $B_d = 0.97$

4.3.5 High-stress, Dynamic-loading, Un-drained Ring-shear Test

If the initial landslide mass were to override torrent bed deposits, an undrained loading process may generate a high pore-water pressure within the torrent deposits and this would help incorporate those deposits into the moving mass (Sassa, et al. 2004a). To examine this scenario for Haivan landslide, an additional sample (S4) was collected from a depth of 5.6 to 6.0 m. The peak friction angles of the three

samples were around 40° (Figure 4.8), so the sample S4 was assumed to have as similar value. To reproduce the stress acting on the sliding surface (around 6 m deep and on a 15° slope), the sample S4 was consolidated to 100 kPa of normal stress and 27 kPa of shear stress. If the displaced mass were to move over the lower slope, the normal stress would increase to 500 kPa corresponding to a depth of 28 m, and the dynamic stresses caused by the displaced mass would be around 420 kPa ($500 \text{ kPa} \times \tan(40^\circ)$). The sliding mass may move quickly, so the test was carried out under the undrained condition. The result of the undrained dynamic loading test is shown in Figure 4.11. When shear stress increases by 37 kPa, the lower slope is sheared and moves together with the original sliding mass. The peak friction angle and the kinetic friction angle were 36.9° . The steady-state shear resistance was 22 kPa.

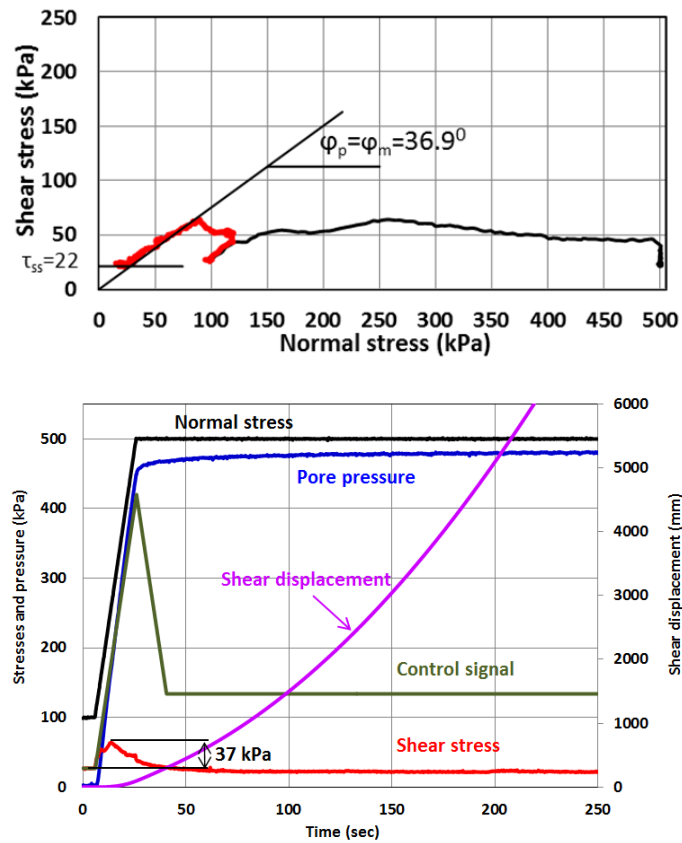


Figure 4.11 Undrained dynamic loading test on sample S4 (6 m). $B_d=0.98$, initial stresses ($\sigma_0=100 \text{ kPa}$, $\tau_0=27 \text{ kPa}$)

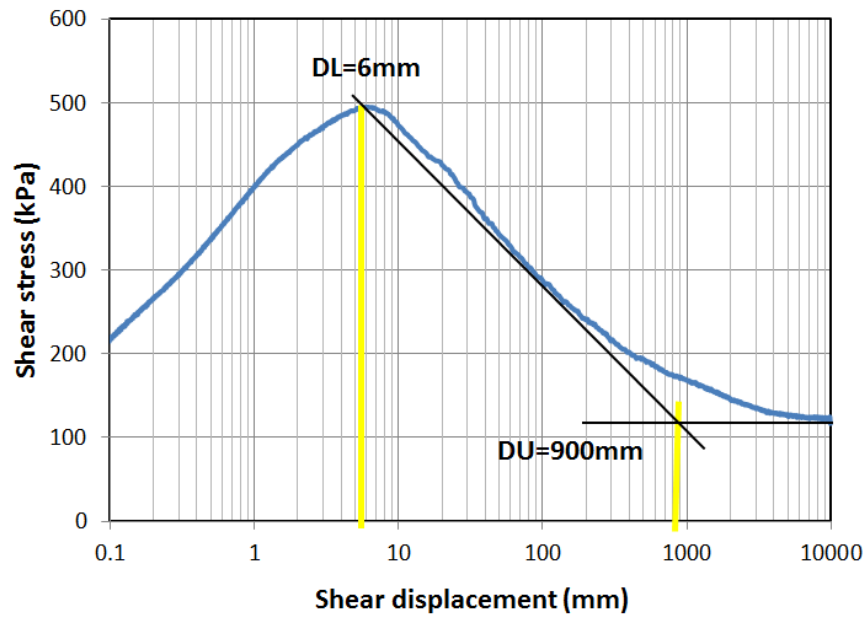


Figure 4.12 Shear displacement and shear resistance path of undrained stress control tests on drill-core sample S1 (50 m)

4.4 Application of the Computer Simulation Model (LS-RAPID) for Hazard Assessment of the Precursor Stage of a Landslide

4.4.1 Landslide Dynamics Parameters

The Haivan Station area is not prone to earthquakes and landslides there are mostly triggered by rain. A simulation of the Haivan precursor landslide was conducted by increasing the pore-water pressure ratio (r_u) as the triggering factor. The shape and depth of the landslide source mass were determined from the interpretation of the results of drilling and field investigation and the assumption of an ellipsoid sliding surface. For the extent of the landslide moving area, it was assumed that a 6.0-m-deep surface deposit was movable when the moving landslide mass reached and shear failure occurred. The blue dotted area in Figure 4.13 shows the extent of the deposit. The parameters used in LS-RAPID were obtained mostly from the undrained dynamic loading ring-shear test results and field investigation. These data are listed in Table 4.2. The landslide dynamic parameters measured by the undrained loading test are shown in Figure 4.11.

Table 4.2. Parameters of Haivan landslide used in LS-RAPID simulation

Parameters	Value	Source
Parameters of Soil in the source area (deeper area)		
Steady-state shear resistance (τ_{ss} , kPa)	116	Test data
Lateral pressure ratio ($k=\sigma_h/\sigma_v$)	0.6-0.7	Estimated
Friction angle at peak (ϕ_p , degree)	39.8	Test data
Cohesion at peak (c , kPa)	20	Estimated
Friction angle during motion (ϕ_m , degree)	38.0	Test data
Shear displacement at the start of strength reduction (DL, mm)	6	Test data
Shear displacement at the start of steady state (DU, mm)	900	Test data
Pore pressure generation rate (B_{ss})	0.7-0.9	Estimated
Total unit weight of the mass (γ_t , kN/m ³)	19	Test data
Parameters of Soil in the moving area (shallower area)		
Steady-state shear resistance (τ_{ss} , kPa)	22	Test data
Lateral pressure ratio ($k=\sigma_h/\sigma_v$)	0.8-0.9	Estimated
Friction angle at peak (ϕ_p , degree)	39.8	Test data
Cohesion at peak (c , kPa)	20	Estimated
Friction angle during motion (ϕ_m , degree)	38.0	Test data
Pore pressure generation rate (B_{ss})	0.7-0.9	Estimated
Total unit weight of the mass (γ_t , kN/m ³)	19.0	Test data
Triggering factor		
Pore pressure ratio changing during rain in the potential shear zone (r_u)	0-0.46	Test data
Other factors		
Slope angle (θ , degree)	20-25	Investigated
Unit weight of water (γ_w , kN/m ³)	9.8	Normal value

(1) The steady-state shear strength (τ_{ss}) of sample S1 obtained from the undrained stress-control test under normal stress of 780 kPa (Figure 4.9A) was 116 kPa. For the landslide moving area τ_{ss} = 22 kPa (Figure 4.11).

- (2) The peak friction angle (ϕ_p) obtained from the undrained stress-control test under a normal stress of 780 kPa (Figure 4.9A) was 39.8°. The friction angle during motion (ϕ_m) was 38°.
- (3) The lateral pressure ratio (k) is in the range from 0.4 to 0.9. The ratio was set to be 0.8 – 0.9 in the moving area. It was 0.6 – 0.7 in the source area and 0.4 outside of the landslide.
- (4) The shear displacement to achieve shear-strength reduction was estimated from the undrained shear-stress control test. Figure 4.12 showed that the critical shear displacement for starting strength reduction (DL) was 6 mm and the beginning of steady state (DU) was 900 mm.
- (5) The pore-pressure generation rate (B_{ss}) was 0.7 – 0.9 in the source and moving area. Outside of the landslide, it was 0.3 (unsaturated).
- (6) The total unit weight of the sample was 21.2 kN/m³ at 1 MPa of normal stress. In shallower areas, the value would be smaller. A single value of 19 kN/m³ was adopted for the entire area.
- (7) The pore-water pressure controlled ring-shear test on sample S1 (Figure 4.10A) indicated that a pore-pressure ratio $r_u=0.46$ could trigger the landslide at Haivan. Therefore, the value of r_u was increased from 0 to 0.50 in the simulation as the triggering factor of the landslide.

4.4.2 Simulation the Landslide Motion-the Case of Haivan Landslide

Based on the lithological descriptions (Figure 4.2), pore-water pressure test results (Figure 4.10), and inclinometer measurements (Figure 4.4), a landslide sliding surface at around 50 m was recognized as potentially the most serious case. Therefore, a landslide integrated numerical model simulation software, LS-RAPID was used in association with the test results on drilling core sample S1 at the depth of 50 m.

The initiation process and the motion process were firstly simulated within the same model from an initially stable state until deposition. Steady-state shear resistance was the key parameter, which was obtained from the ring-shear test results. Under a given pore-water ratio (r_u), the model simulated a potential landslide triggered by seismic load. During the motion, landslide volume and hazard area were increased by the entrainment of unstable deposits. The LS-RAPID model is based on the assumption that the frictional energy at the sliding surface is consumed potential

energy. In addition, the model provides a non-frictional energy loss function for a specific mass to simulate kinetic energy consumption during collision of sub-masses within a landslide mass Gradiski, et al., 2013.

In the LS-RAPID modeling framework, the landslide topography of Haivan Station area was represented by a digital surface model (DSM). The simulation model covered the orthogonal matrix 1740 m x 1650 m in x and y directions (Figure 4.13), which was divided into square grid cells of 30 m x 30 m. The points inside the polygon have the real elevations of 590 m and 0 m, respectively for highest and lowest levels. Necessary parameters for simulation were determined including steady-state shear strength (τ_{ss}), the rate of excess pore-water pressure generation, peak friction angle (ϕ_p) and lateral pressure ratio (k).

To assess the hazard of the Haivan Station landslide, four simulations were performed with different values of pore-water pressure ratio (0.20, 0.30, 0.45 and 0.46). In the case of $r_u=0.20$, the pore-pressure ratio was increased from $r_u=0.0$ at the beginning of the simulation to $r_u=0.20$, with $\Delta r_u=0.01/s$ and held constant when it reached $r_u=0.20$. The cases with $r_u=0.30$, 0.45, and 0.46 were conducted similarly to the $r_u=0.20$ case. The computer simulations were run until a zero velocity was obtained for all meshes (following Sassa, et al. 2010). The landslide simulation results are presented in Figure 4.13. The yellow line is the potential landslide boulder, the pink line is the calculated sliding-mass distribution area, the blue coloured balls present the current state of the Haivan slope, and the red coloured balls represent the moving mass. In the simulation, there was no landslide when the pore-pressure ratio r_u was 0.20 (Figure 4.13a). Figure 4.13b and 13c present the cases in which local failures were simulated, but without further progressive failure. In the case of the maximum $r_u=0.30$ (Figure 4.13b), local failures occurred but they were limited, mostly to the right-hand side of the slope. When r_u reached 0.45 (Figure 4.13c), local failures occurred at the right side, in the central and in the upper part of the slope. When r_u was given as 0.46 (Figure 4.13d), the whole landslide mass was formed and it moved over the railway. The total volume of this simulated landslide would be $3.53 \times 10^6 \text{ m}^3$.

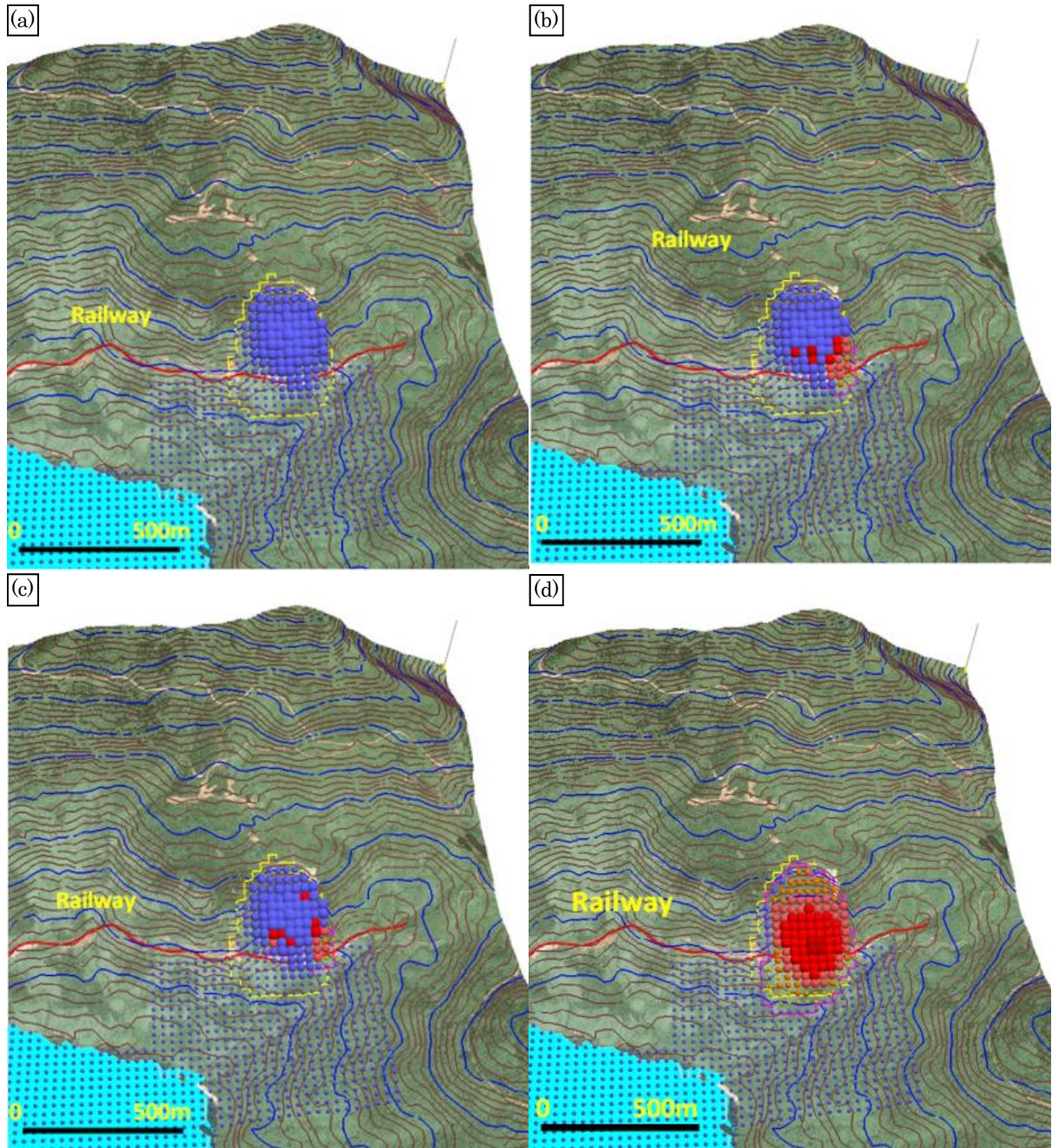


Figure 4.13 Landslide deposits with pore-pressure ratio (r_u) equal to (a) 0.20, (b) 0.30, (c) 0.45 and (d) 0.46

Figure 4.10A shows that failure could occur if the pore-water pressure increased up to 360 kPa, a pore-water pressure ratio $r_u=0.460$ for the sample at 50 m depth. In addition, the simulation results showed that local failure occurred when $r_u=0.450$ and a landslide was formed when $r_u=0.460$. Therefore, a value of pore-water pressure ratio $r_u=0.460$ would be expected to cause failure of the landslide. To study the landslide development process in detail and assess the exact value, another computer simulation was performed with pore-water pressure ratio increasing from $r_u=0.450$ at the beginning of the simulation to $r_u=0.460$ over 100 seconds (the rate of

increase in the pore-water pressure ratio was 0.0001/s) and then held constant at $r_u=0.460$. The result is shown in 0a, b, c, d. At 1 s, the pore-water pressure reaches 0.4501, failure starts in the middle part of the slope and at the right side of the slope. At 10 s, r_u reaches 0.4510, and failure spreads to the lower half of the slope, with a velocity of 2.2 m/s. At 27 s, r_u reaches 0.4527 and the whole landslide mass is formed and moves with a maximum speed of 4.4 m/s. At 50 s, the landslide mass stops and is deposited. 0e shows the relationship between time and pore-water pressure ratio and speed of the mass movement.

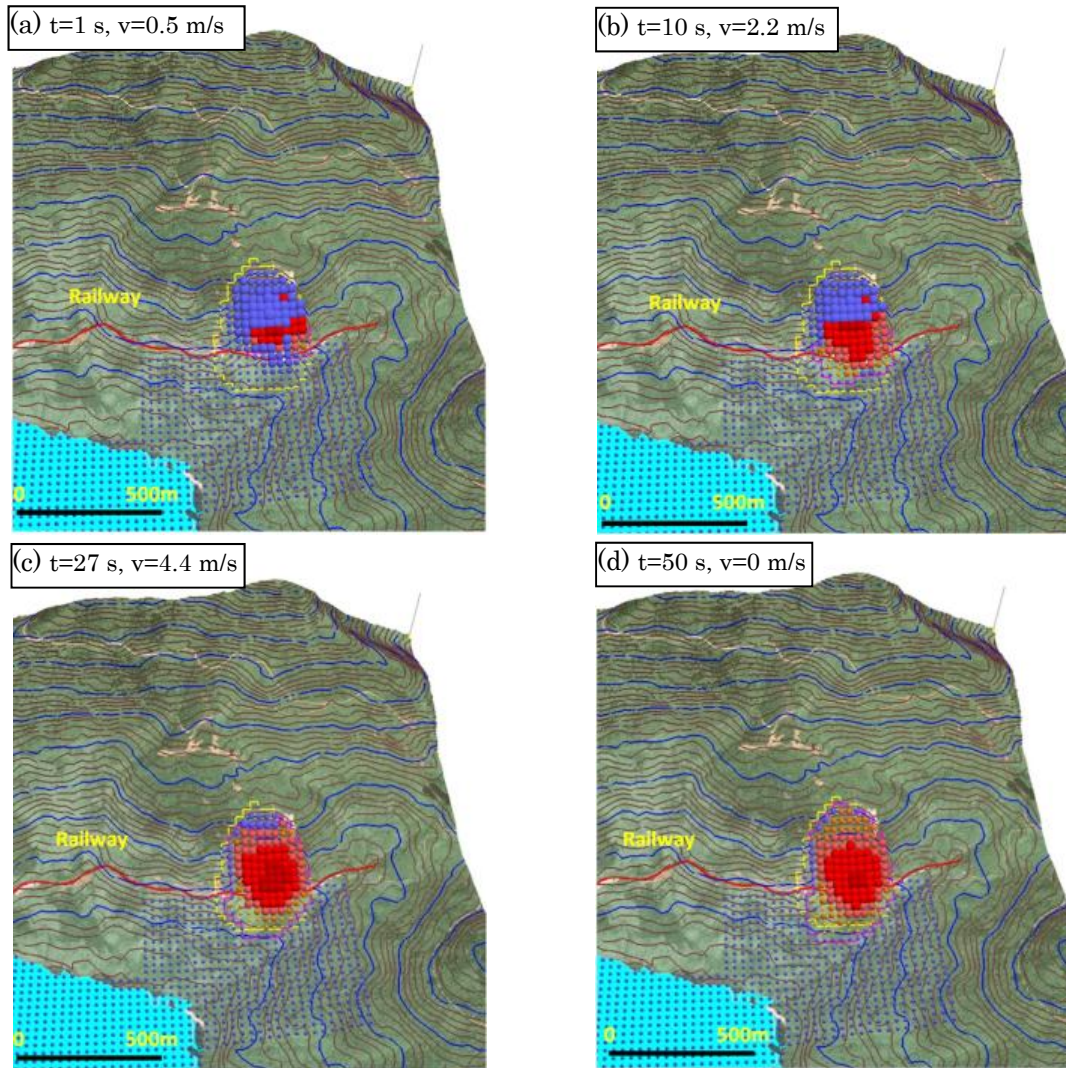


Figure 4.14 Landslide development process $\Delta r_u=0.0001/s$. (a) $r_u=0.4501$, $t=1$ s; (b) $r_u=0.4510$, $t=10$ s; (c) $r_u=0.4527$, $t=27$ s; (d) $r_u=0.4550$, $t=50$ s)

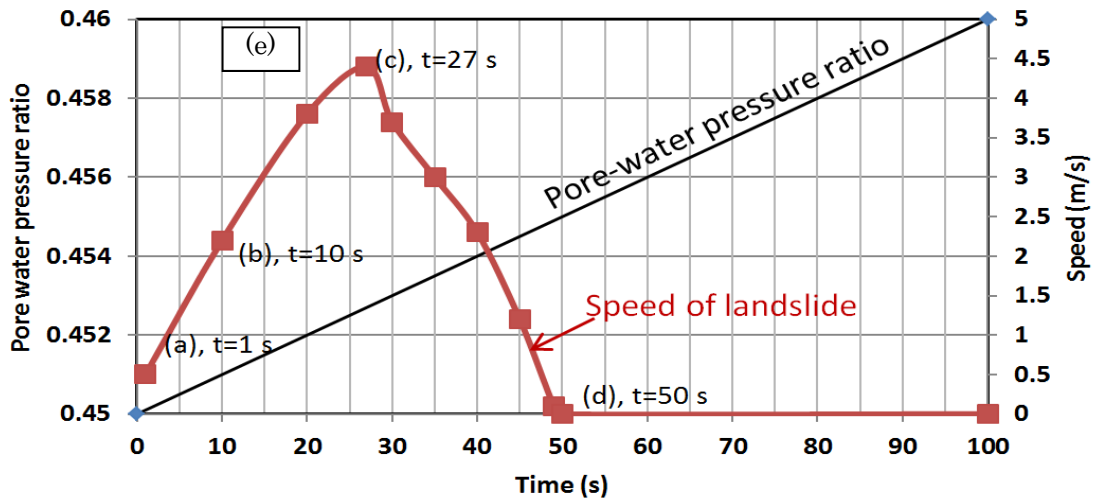


Figure 4.14 (continued) (e) pore-water pressure ratio and speed of landslide in relation to time

4.4.3 Landslide Hazard Assessment for Haivan Station

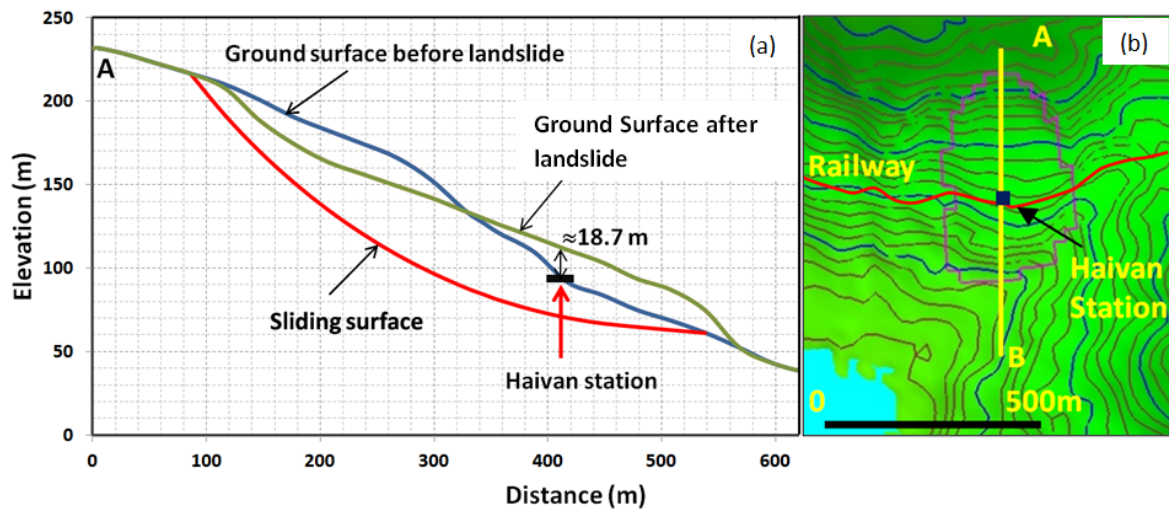


Figure 4.15 Hazard assessment of the precursor stage of the potential landslide at Haivan station. (a) A-B cross section of the slope before and after the possibly triggered landslide; (b) Landslide hazard area by the LS-RAPID simulation (within the pink color line).

Results of hazard assessment of the precursor-stage landslide in the slope including the Haivan Station is shown in Figure 4.15. This figure presents the plan of landslide affected area and the section of landslide movement assessed by LS-RAPID as the pore-pressure ratio reached 0.460 (Figure 4.13d). Figure 4.15a presents the section of ground surface before and after the assessed landslide. The ground of the Haivan Station would be covered by 18.7 m thickness of landslide debris. It would be something similar to photo Figure 4.1b, but much bigger. Figure

4.15b presents the whole landslide-affected area including the source area and the moving area.

The critical pore-water pressure working on the sliding surface to initiate the landslide can be estimated. 0 presents: (a) a local failure in the slope; (b) progressive failure to create a landslide block; (c) further development of the landslide block and its motion; and (d) the termination of movement. At the stage of local failure (a), slope deformation may stop without reaching the stage of motion of the larger landslide. However, the stage of (b) may move as a landslide block, although the whole landslide block has yet to be made. The point at which the pore-pressure ratio reached 0.451 (2.0 m/s in speed) was regarded to mark the initiation of landslide movement. The speed of 4.4 m/s in 0 was the maximum velocity in a 30 m mesh unit of landslide mass. It does not mean the average velocity of the whole landslide mass.

The pore-water pressure acting on the sliding surface at 50 m was (50 m – the monitored depth of water table within BH1) x (the unit weight of water).

The normal stress (σ) acting on the sliding surface was calculated as $\sigma = \gamma_t \times h_s \times \cos^2 \theta$ where γ_t is the total unit weight of soil mass, h_s is the depth of soil above the sliding surface, and θ is the inclination of the sliding surface at this point.

The landslide will initiate in an unstable part, namely a part with a greater inclination of sliding surface. The local inclination of the sliding surface is currently unknown. It was assumed in this analysis to lie between two values: one is the inclination (20°) of a line between the possible head scarp (P₁, Figure 4.16) and the Haivan Station (P₂, Figure 4.16); the other (25°) is the inclination between the step (P₁) and the point along the sliding surface just below the initial failure point in 0 (a) (PF, Figure 4.16), very close to BH-1. The critical pore-pressure ratio of 0.451 is u/σ .

From these, the critical depths of water table in BH-1 were calculated as 11.4 m (for 20°) and 14.1 m (for 25°) as shown in Figure 4.16.

The current monitoring period is only 14-months long. The highest water depth reached in this period was 15.2 m. If the monitored water level in BH-1 reach higher than 14.0 m, it could initiate a landslide. So, water-level monitoring can be a tool for early warning.

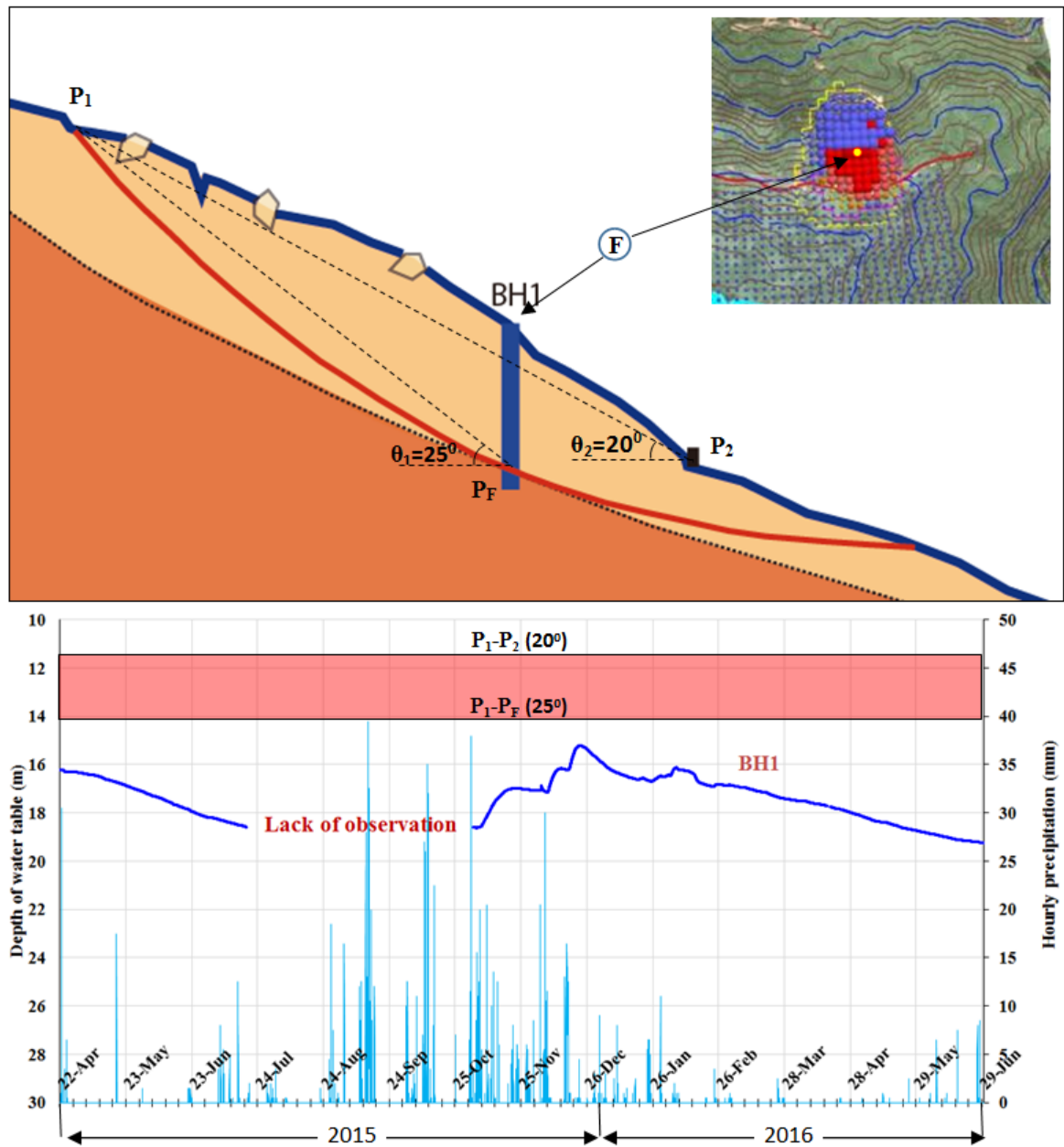


Figure 4.16 Critical condition of landslide initiation as a tool for early warning of landslides

4.5 Summarization

This chapter describes the first application of the undrained dynamic loading ring-shear apparatus to drill-core samples for assessing the hazard of the precursor stage of a landslide. The precursor stage of a large sliding block has formed in the Haivan Station area following a series of previous small landslides. Comparison between the analysis using the LS-RAPID software and the monitoring of the depth to the water table in BH-1 could provide a tool for early warning of this landslide (Figure 4.16).

Chapter 5 Conclusions

The initiation and motion of rapid landslides are complicated processes. The mechanism of rapid landslide is proposed as for a slope at critical stress condition, with an external disturbance such as earthquake or rainfall or combination both factors, the soil at sliding surface would fail. If this process performs with excess pore-pressure generation, and changes in grain size, it leads to a rapid landslide occurrence.

ICL-1 and ICL-2 were developed as a part of Japan-Croatia and Japan-Vietnam research projects, respectively. Two apparatus were created to simulate the initiation and motion of rainfall and earthquake-induced rapid landslide. Two real cases of rapid landslide were applied to ICL-1 and ICL-2. The first is the 28 July 2015 Rapid Landslide at Ha Long City, Quang Ninh, Vietnam triggered by heavy rainfall and the second is landslides induced by the 16 April 2016 Kumamoto Earthquake, Japan. A series of tests were conducted on the samples taken from two landslides to examine the mechanism of rainfall and earthquake-induced rapid landslides as well as to simulate initiation and motion of the landslides. All parameters measured from the undrained ring-shear tests were inputted to the integrated landslide simulation model (LS-RAPID) as dynamic landslide parameters.

1. By the ring shear test results and Ha Long landslide simulation results using LS-RAPID, we conclude as following:

- The Ha Long landslide was small but it caused a great disaster. The landslide occurred in an area where high soil permeability soil (brown layer) overlies lower soil permeability layer (black layer). During heavy rain, the low permeability of the black layer soil allows high pore-water pressures to build up in the brown soil layer above resulting in the Ha Long landslide failure.
- This research has successfully demonstrated rapid flow failure occurs through the liquefaction of the landslide shear zone due to rapid pore-water pressure rise replicated in the ring shear testing. The SLIDE model is useful to calculate pore-water pressure from rainfall intensity for use in shallow-landslide models. The LS-RAPID model produced a landslide area that was broadly consistent with field

investigation results. In addition, the modeled time of landslide occurrence is close to that reported by local inhabitants.

- The researched results contribute to understand the mechanism of a rapid landslide during heavy rain as a basic knowledge for disaster prevention. It may provide useful information for the local government authorities in Ha Long city for land-use planning in the hilly areas of the city.

2. By the ring shear test results and Kuramoto landslide simulation results using LS-RAPID, the conclusion is:

An undrained, dynamic-loading ring-shear apparatus (ICL-2) was used to study two rapid and long-runout landslides triggered by the largest earthquake of the April 2016 Kumamoto earthquake series in Kumamoto Prefecture. The undrained seismic shear stress loading test with the simulated earthquake wave suggested that the mechanism of the rapid and long runout motion of the two major landslides in Minamiaso village was due to “sliding surface liquefaction” (Sassa, et al. 2014b). The experimental result also suggested that the landslides could have been triggered by a weaker earthquake. Parameters obtained from field investigation and laboratory experiments were used in a landslide computer simulation (LS-RAPID). Pore-pressure ratio $r_u=0.2$ (for Aso-ohashi landslide) and $r_u=0.4$ (for Takanodai landslide) and the seismic acceleration of the M7.3 Kumamoto earthquake wave recorded at the KMM005 station were used as triggering factors. The computer simulation (LS-RAPID) reproduced the landslides with a similar travel distance and distribution of mass to that shown in the UAV images. Although the Takanodai slope has a sliding surface with an inclination of only about 11 degrees, the landslide moved over a wide area and a long-distance due to the high fluidized state of the soil mass.

3. Following two real case-studies of rapid landslides, the combination of ring shear apparatus and LS-RAPID was applied to a precursor stage of a landslide at the Haivan station, Vietnam. This is the first application of the undrained dynamic loading ring-shear apparatus to drill-core samples for hazard the susceptibility of a precursor stage of a landslide. The conclusions are listed as follows:

- A precursor stage of a large sliding block has formed in the Haivan Station area following a series of previous small landslides. Comparison between the analysis using the LS-RAPID software and the monitoring of the depth to the water table in BH-1 could provide a tool for early warning of this landslide (Figure 4.16). The

landslide could be triggered if the pore-water pressure at the current precursor stage of sliding surface rises to the value equivalent to groundwater level at 14.1 m.

- Undrained tests on the core samples with the ring-shear apparatus ICL-2 were successful in a monotonic speed-controlled test, a monotonic stress-controlled test, and a pore-pressure controlled test.

- All the simulation results suggested that a local landslide could occur from the middle point of the slope if the pore-water pressure r_u reached a value of 0.3. However, a large Haivan Station landslide mass could be formed if r_u reached 0.4527. It could reach a maximum speed of 4.4 m/s and destroy the national railway line at this site. The total volume of such a landslide was estimated to about $3.5 \times 10^6 \text{ m}^3$. The overburden would be about 18.7 m thick covering the railway.

References

- Arbanas, Z., Arbanas, S., Vivoda, M., Peranic, J., Jovancevic, S., & Jagodnik, V. (2014). Identification, monitoring and simulation of landslides in the Rjecina River Valley, Croatia. Proceeding of SATREPS 2014 Workshop “Landslide Risk Assessment Technology”, 29-30 July 2014 in Hanoi, Vietnam, pp 200-213.
- Baum, R., Schulz, W., Godt, J., Godt, D., Ried, M., & Burns, W. (2014). Plenary: progress in regional landslide hazard assessment - Examples from USA. Landslide Science for a Safer Geoenvironment. (Sassa, Canuti, Yin eds), Vol.1, pp.21-36.
- Bishop, A., Green, G., Garge, V., Andersen, A., & Brown, J. (1971). A new ring-shear apparatus and its application to the measurement of residual strength. *Geotechnique* 21, pp 273–328.
- Boldini, D., Wang, F., Sassa, K., & Tommasi, P. (2009). Application of large-scale ring shear tests to the analysis of tsunamigenic landslides at the Stromboli volcano, Italy. *Landslides* 6 (3): 231–240.
- Casagrande. (1949). A oil Mechanics in the design and Construction of the Logan Airport. *J. Boston Soc. Civil Eng.*, Vol 36, No. 2. pp. 192–221.
- Central Committee for National Disaster Prevention and Control (CCNDPC-Vietnam). (2015). Report on natural disaster in Quang Ninh. [http://www.dmc.gov.vn/chi-tiet-thien-tai/mua-lu-mien-bac-viet-nam-lu-tai-quang-ninh-dis156.html?lang=vi-VN#prettyDetail\[gallery1\]/2/](http://www.dmc.gov.vn/chi-tiet-thien-tai/mua-lu-mien-bac-viet-nam-lu-tai-quang-ninh-dis156.html?lang=vi-VN#prettyDetail[gallery1]/2/).
- Dang, K. (2015). Development of a new high-stress dynamic-loading ring-shear apparatus and its application to large-scale landslides. Doctoral Thesis, Graduate School of Engineering, Kyoto University. 79 pages.
- Dang, K., Sassa, K., Fukuoka, H., Sakai, N., Sato, Y., Takara, K., Lam, H., Loi, D., & Tien, PV., Ha, ND. (2016). 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* 13(6): 1525–1534.

- Duc, DM. (2013). Rainfall-triggered large landslides on 15 December 2005 in Van Canh District, Binh Dinh Province, Vietnam. *Landslides* 10(2):219–230.
- Fukuda, M., Kanai, Y., Shinkai, A., & Sakai, Y. (2002). Prediction Method for Landslide using a Vibratory Sensor and a Low-Frequency Amplifier (in Japanese). Proceedings of 37th annual meeting of the Japan Geotechnical Society, pp.2247-2248.
- Fukuoka, H., Wang, G., Sassa, K., Wang, F., & Masumoto, T. (2004). Earthquake-induced rapid long-traveling flow phenomenon: May 2003 Tsukidate landslide in Japan. *Landslide* 1(2):151–155.
- Furumura, T. (2016). Destructive near-fault strong ground motion from the 2016 Kumamoto Prefecture, Japan, M7.3 earthquake, *Landslides* (accepted).
- Geological map display system of Geological Survey of Japan, AIST. (2016). Geological map of Aso volcano. <https://gbank.gsj.jp/geonavi/geonavi.php#latlon/13,32.88556,131.07891>.
- Gradiski, K., Sassa, K., He, B., Arbanas, Z., Arbanas, S., Krkac, M., Kvasnicka, P., & Ostric, M. (2013). Application of Integrated Landslide Simulation Model LS-Rapid to the Kostanjek Landslide, Zagreb, Croatia. Proceedings of the 1st Regional Symposium on Landslides in the Adriatic Balkan Region, pp 11-16.
- Han-Balder, H., & Celine, B. (2010). Earthquake - induced landslide hazards in mountain regions: A review of case histories from central ASIAN. *Geologica belgica*: 135-150.
- He, B., Sassa, K., Nagai, O., & Takara, K. (2014). Manual of LS-RAPID numerical simulation model for landslide teaching and research. Proceedings of the 1st Regional Symposium on Landslides in the Adriatic Balkan Region, pp. 193–197.
- http://en.wikipedia.org/wiki/1972_Unzen_earthquake_and_tsunami. (n.d.).
- http://en.wikipedia.org/wiki/Lisi_of_landslides. (n.d.).
- <http://landslides.usgs.gov/learn/majorls.php>. (n.d.).
- Hung, L., Van, N., Duc, D., Ha, L., Son, P., Khanh, NH., Binh, LT. (2016). Landslide susceptibility mapping by combining the analytical hierarchy

- process and weighted linear combination methods: a case study in the upper Lo River catchment (Vietnam). *Landslides* 13(5): 1285–1301.
- International Consortium on Landslide. (2016). Instruction for authors of world reports on landslides. <http://iplhq.org/icl/wp-content/uploads/2016/02/1.-Instruction-for-WRLFfinal.doc>.
- Kirschbaum, D., Stanley, T., & Zhou, Y. (2015). Spatial and temporal analysis of a global landslide catalog. *Geomorphology*, Volume 249, pp 4-15.
- Kjekstad, O., & Highland, O. (2009). Economic and Social Impacts of Landslides. *Landslides*. doi:10.1007/978-3-540-69970-5_30.
- Kubota, Y. (2003). Some indication before the landslide, Sumikawa area, Kazuno, Akita Prefecture, 1997 (in Japanese). *Journal of the Japan Landslide Society*, Vol.40(1), pp.84-87.
- Lagmay, A. e. (2006). Scientists investigate recent Philippine landslide. *Eos*. 87(12).pp. 121–124.
- Lam, H., Dang, K., Tien, PV., V., Loi, D., & Thanh, NK. (2014). Recent development of the new high stress undrained ring-shear apparatus (ICL-2) and its application. *Proceeding of SATREPS 2014 Workshop “Landslide Risk Assessment Technology”*, 29-30 July 2014 in Hanoi, Vietnam, pp 168-173.
- Lam, H., Loi, D., Sassa, K., Takara, K., Ochiai, H., Dang, K., Abe, S., Asano, S., & Ha, D. (2017). Susceptibility Assessment of a Precursor Stage of Landslide Threatening the Haivan Railway Station in Vietnam. *Landslides*. doi: 10.1007/s10346-017-0870-3.
- Liao, Z., Hong, Y., Kirschbaum, D., & Liu, C. (2012). Assessment of shallow landslides from Hurricane Mitch in central America using a physically based model. *Environmental Earth Sciences* 55:1697-1705.
- Liao, Z., Hong, Y., Wang, J., Fukuoka, H., Sassa, K., Karnawati, D., & Fatani, F. (2010). Prototyping an experimental early warning system for rainfall induced landslides in Indonesia using satellite remote sensing and geospatial datasets. *Landslides* 7(3): 317-324.
- Loi, D., Lam, H., Sassa, K., Takara, K., Dang, K., Thanh, NK., & Tien, PV. (2017). The 28 July 2015 rapid landslide at Ha Long City, Quang Ninh, Vietnam. *Landslides*. doi:10.1007/s10346-017-0814-y.

- Marijan, C., Danijela, J. K., & Meho - Sasa, K. (2016). Application of unmanned aerial vehicle for landslide mapping. *New technologies in engineering geodesy and multisensor systems*.
- Montarasio, L., & Valentino, R. (2008). A model for triggering mechanisms of shallow landslides. *Natural Hazards and Earth Sciences* 8:1149-1159.
- Nakano, N., Osanai, Y., Owada, M., Nam, T., Charusiri, P., & Khamphavong, K. (2013). Tectonic evolution of high-grade metamorphic terranes in central Vietnam: Constraints from large-scale monazite geochronology. *Journal of Asian Earth Sciences* 73 (2013) pp 520–539.
- Okada, Y., Ochiai, H., Okamoto, T., Sassa, K., Fukuka, H., & Igwe, O. (2007). A complex earth slide—earth flow induction by the heavy rainfall in July 2006, Okaya City, Nagano Prefecture, Japan. *Landslides* 4(2):197–203.
- Sassa, K., & He, B. (2013). TXT-Tool 3.081-1.1 Landslide initiation mechanism. *ICL Landslide Tools*:215-237.
- Sassa, K., Bin, H., Dang, K., Nagai, O., & Takara, K. (2014b). Plenary: Progress in Landslide Dynamics. *Landslide science for a safer geoenvironment*, Vol. 1, Springer, pp 37-67.
- Sassa, K., Dang, K., He, B., Takara, K., Inoue, K., & Nagai, O. (2014a). A new high-stress undrained ring-shear apparatus and its application to the 1792 Unzen–Mayuyama megaslide in Japan. *Landslides* 11 (5), pp 827-842.
- Sassa, K., Fukuoka, H., Wang, F., & Wang, G. (2005). Dynamic properties of earthquake-induced large-scale rapid landslides within past landslide masses. *Landslides* 2:125–134.
- Sassa, K., Fukuoka, H., Wang, G., & Ishikawa, N. (2004a). Undrained dynamic-loading ring-shear apparatus and its application to landslide dynamics. *Landslides* 1 (1), pp 7–19.
- Sassa, K., He, B., Dang, K., & Nagai, O. (2014). Progress in landslide dynamics. *Landslide science for a safer geoenvironment*. Proc. the third world landslide forum, springer. Vol 1:37–67.
- Sassa, K., He, B., Miyagi, T., Konagai, K., Ostric, M., Setiawan, H., Takara, K., Nagai, O., Yamashiki, K., & Tutumi, S. (2012). A hypothesis of the Senoumi submarine megaslide in Suruga Bay in Japan—based on the undrained

- dynamic-loading ring shear tests and computer simulation. *Landslides* 9 (4), pp 439–455.
- Sassa, K., Nagai, O., Solidum, R., Yamazaki, Y., & Ohta, H. (2010). An integrated model simulating the initiation and motion of earthquake and rain induced rapid landslides and its application to the 2006 Leyte landslide. *Landslides* 7 (3), pp 219–236.
- Sassa, K., Wang, G., Fukuoka, H., Wang, F., Ochiai, T., Sugiyama, M., & Sekiguchi, T. (2004b). Landslide risk evaluation and hazard zoning for rapid and long-travel landslides in urban development areas. *Landslides* 1 (3), pp 221–235.
- Shinro, A., Masao, Y., Kumiko, F., Mauri, M., Eileen , M., Osamu, N., Bin, H., Duc, DM., Wang, F., & Ochiai, H. (2013). TXT-tool 2.081-1.1 Key points in field work for Landslide engineers. *ICL Landslide Teaching Tools*: 89-145.
- ShouBiao, Z., YaoLin, S., Ming, L., & FuRen, X. (2013). Dynamic mechanisms of earthquake - triggered landslides. *Science China Earth Sciences*, Volume 56, Issue, pp 1769-1779.
- Tam, D. (2005). Report on the causes of slope failure at Haivan station and a proposal of countermeasures. Research project in transportation sector, Ministry of Transport, 18 pages (in Vietnamese).
- The Japan Time. (2016). Kyushu villagers trapped by landslides, severed roads. [http:// www.japantimes.co.jp/news/2016/04/16/national/kyushu-villagers-trapped-by-landslides-severed-roads/#.V3DILPI95D8](http://www.japantimes.co.jp/news/2016/04/16/national/kyushu-villagers-trapped-by-landslides-severed-roads/#.V3DILPI95D8).
- The Mainichi newspapers. (2016). Earthquake-induced landslides in Kumamoto village unexpected by national standards.
- Tien, B. D., Pradhan, B., Lofman, O., Revhaug, I., & Dick, O. (2013). Regional prediction of landslide hazard using probability analysis of intense rainfall in the Hoa Binh Province, Vietnam. *Nat Hazards* 66:707–730.
- Tien, B. D., Tuan, T., Hoang, N., Thanh, N., Liem, N., & Pradhn, B. (2016). Spatial prediction of rainfall-induced landslides for the Lao Cai area (Vietnam) using a hybrid intelligent approach of least squares support vector machines inference model and artificial bee colony optimization. *Landslides*. doi:10.1007/s10346-016-0711-9.

- Tien, P., Sassa, K., Takara, K., Binh, H., & Luong, L. (2015). Characteristics and Failure mechanism of landslides in weathered granitic rocks in Haivan mountain. Proceeding of International conference on landslide and slope stability 2015”, pp 165-172.
- Tonje, G., & Henrik, J. (2016). Assessment of data availability for the development of landslide fatality curves. *Landslides*. doi 10.1007/s10346-016-0775-6.
- Wang, G., Suemine, A., Furuya, G., Kaibori, M., & Sassa, K. (2005). Rainstorm-induced landslides at Kisawa village, Tokushima Prefecture, Japan, August 2004. *Landslides* 2(3):235–242.
- Yin, Y., Xing, A., Wang, G., Feng, Z., Li, B., & Jiang, Y. (2016). Experimental and numerical investigations of a catastrophic long-runout landslide in Zhenxiong, Yunnan, southwestern China. *Landslides* (2016). doi:10.1007/s10346-016-0729-z.
- Zhang, Z. (1995). Geological Disasters in Loess Areas during the 1920 Haiyuan Earthquake, China. *GeoJournal*, 36.2, 3: 269-274.