

**Mitigation of dip slope collapse due to
groundwater infiltration using
counterweight fill and slope stability
analysis considering lateral confinement**

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Abstract

A slope lying on bedding, often referred to as a dip slope, is a geological formation where the slope surface parallels or closely follows the orientation of underlying sedimentary rock layers or bedding planes. Dip slopes have a potential for landslides, as evidenced by past incidents where landslides have caused damage to infrastructure and posed risks to human safety. This thesis focuses on the stability analysis of dip slopes under varying conditions, with an emphasis on groundwater infiltration and seismic activity. The primary objective is to develop theoretical models and conduct physical experiments to understand the failure mechanisms and improve the assessment of dip slope stability.

A physical model of a dip slope was designed and utilized to investigate failure mechanisms under controlled conditions. Experiments employed both 1g and centrifuge setups to simulate realistic scenarios and evaluate the impact of different factors on slope stability. The reverse toe angle emerged as a critical factor influencing failure modes, with distinct behaviors such as toe sliding and thrust failure identified and analyzed using Particle Image Velocimetry (PIV) techniques. Additionally, a novel minimum coefficient of variation method was developed to analyze PIV results and detect slip line formations. The centrifugal model was newly designed to incorporate the effect of groundwater infiltration.

Theoretical analyses were developed to complement experimental findings, focusing on assessing dip slope stability under groundwater infiltration. Analysis of dip slope stability was enhanced through considerations of lateral confinement, transitioning from 2-dimensional to 3-dimensional analyses. Results indicated a strong correlation between dip slope stability analyses with lateral confinement and experimental outcomes.

Furthermore, counterweight fill was investigated as an effective measure to mitigate slope instability. Theoretical predictions and experimental observations affirmed that counterweight fill shifts failure modes from toe sliding to thrust failure, thereby enhancing slope stability across varied conditions. Moreover, dip slope stability analysis with lateral confinement was utilized to assess failure modes post-counterweight fill.

In conclusion, this thesis advances the understanding of dip slope stability through integrated theoretical analyses and physical modeling. The insights gained contribute to mitigating risks associated with dip slope landslides, thereby supporting more informed geotechnical engineering practices.

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Table of Contents

Abstract.....	iii
Acknowledgement.....	iv
Table of Contents	vi
List of Figures.....	ix
List of Tables.....	xvi
Chapter 1 Introduction	1
Background.....	1
Chapter 2 Literature Review	6
2.1 Failure mechanisms of slope lying on bedding plane	6
2.2 Physical modelling of slope lying bedding plane triggered by thrust load	10
2.3 Lateral confinement on slope lying on bedding plane	13
2.4 Physical modelling with groundwater seepage	18
2.5 Scopes and structure.....	23
2.5.1 Statement and problem	23
2.5.2 Objective	24
Chapter 3 Methodology.....	26
3.1 1g physical model	26
3.2 Centrifugal model.....	28
3.2.1 Basic theory and scaling laws in centrifuge modelling	28
3.2.2 Inherent deficiencies on centrifuge modelling.....	30
3.2.3 Centrifuge laboratory at Kyoto University	32
3.2.4 Geotechnical centrifuge on landslide.....	32
3.3 Model apparatus	34
3.4 Measuring instruments	38
3.4.1 Pressure gauge	38
3.4.2 Laser sensor	38
3.4.3 Pore-water pressure transducer	39
3.4.4 Accelerometer	39
3.5 Experimental material	40
3.5.1 1g test.....	40
3.5.2 Centrifuge test.....	41
3.6 Model preparation	43
3.6.1 1g test.....	43
3.6.2 Centrifuge test.....	44
3.7 Particle image velocimetry.....	45
Chapter 4 Study the reverse toe angle effect of dip slope failure mechanism....	48
4.1 Observation of failure	49
4.2 Particle image velocimetry.....	51
4.3 Statistical approach for detection of slip lane	53
4.4 Calculation of failure time by minimum coefficient of variation	55
4.5 Inverse velocity method	64
4.6 Summary	70
Chapter 5 Centrifuge Model of Dip Slope on Impermeable Bedding Plane with Groundwater Infiltration and Passive Thrust Induced Instability	71

5.1	Theoretical calculations of slope lying on bedding plane with lateral and toe supports.	71
5.2	Experimental setup and preparation of centrifuge physical model	77
5.2.1	Instrumentation	78
5.2.2	Preparation of centrifuge physical model	79
5.3	Method	81
5.4	Experimental results	82
5.4.1	Test 1: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$	82
5.4.2	Test 2: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$	84
5.4.3	Test 3: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$	86
5.4.4	Test 4: Geometry condition $\alpha=30$, $\beta=0$, $T = 5\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$	88
5.4.5	Test 5: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$	90
5.4.6	Test 6: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$	92
5.4.7	Experimental summation	94
5.5	PIV analysis	94
5.6	Reduction factor	98
5.7	Discussion	100
5.7.1	Factor of safety (FOS)	100
5.7.2	Failure mode	102
5.7.3	Back-calculation of surcharge weight	103
5.8	Summary	105
Chapter 6 Centrifugal Model for Counterweight Mitigating Dip Slope with Groundwater Seepage		106
6.1	Theoretical calculations of counterweight(berm) mitigation on slope lying bedding plane with lateral and toe supports with groundwater seepage	106
6.2	Experimental setup and preparation of centrifuge physical model	110
6.2.1	Instrumentation	111
6.2.2	Preparation of centrifuge physical model	112
6.3	Method	114
6.4	Experimental result	115
6.4.1	Test 1: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: Type 1	116
6.4.2	Test 2: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: Type 2	118
6.4.3	Test 3: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 1cm of height	120
6.4.4	Test 4: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 1.75cm of height	122
6.4.5	Test 5: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 2cm of height	124
6.5	Reduction factor	126

6.6	PIV analysis	126
6.7	Discussion	130
6.7.1	Factor of safety (FOS)	130
6.7.2	Failure mode	131
6.7.3	Optimum counterweight	133
6.7.4	Back calculation of applied force	133
6.8	Summary	136
Chapter 7 Centrifugal Model of Dip Slope on Permeable Bedding Plane with Groundwater Seepage and Passive Thrust Induced Instability		137
7.1	Experimental setup and preparation of centrifuge physical model	137
7.2	Method	138
7.3	Experimental result	139
7.3.1	Test 1: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$	139
7.3.2	Test 2: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$	141
7.3.3	Test 3: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 1cm of height	143
7.3.4	Test 4: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 2 cm of height	145
7.3.5	Test 5: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 3 cm of height	147
7.3.6	Test 6: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 4 cm of height	149
7.4	Reduction ratio	151
7.5	Discussion	153
7.5.1	Factor of safety (FOS)	153
7.5.2	Failure mode and optimum counterweight	155
7.5.3	Back calculation of applied force	155
7.6	Summary	158
Chapter 8 Pseudo-static Stability Analysis of a Laterally Confined Dip Slope Investigated by Dynamic Centrifugal Model		159
8.1	Theoretical calculations of slope lying on bedding plane with lateral and toe supports subjected to seismic load	159
8.2	Model description.....	162
8.3	Experimental procedures.....	164
8.4	Experimental results.....	165
8.4.1	Failure observations	165
8.4.2	Displacement.....	166
8.4.3	Earth pressure.....	166
8.5	Discussions.....	168
8.6	Summary	170
Chapter 9 Conclusions and Future work		171
References.....		175

List of Figures

Figure 1-1 (a) Photo displaying Freeway no.3 landslide in Taiwan from NASC Taiwan (2010). (b) Photo displaying disaster relief in progress from NASC Taiwan (2010), The picture in the upper right corner of (b) is a close-up view of the pre-disaster slope, which is marked as a yellow dash circle in (a) and (b) (Yeh et al., 2021)	2
Figure 1-2 Photograph of low wall failure (Pipatpongsa et al., 2023)	3
Figure 1-3 Monitoring of surface slope displacement (Pipatpongsa et al., 2023).....	3
Figure 1-4 SE pit and method of preventing slope sliding(Pipatpongsa et al., 2024).....	4
Figure 2-1 Bilinear footwall failure and location of transition zone(Stead & Eberhardt, 1997).....	7
Figure 2-2 Mechanism of ploughing stab failure in foot wall slope(Stead & Eberhardt, 1997).....	7
Figure 2-3 Geometric examination of bilinear slab failure(Alejano et al., 2011)	8
Figure 2-4 Geometric examination of ploughing slab failure (Alejano et al., 2011)	9
Figure 2-5 Simulation results of footwall analysis (Havaej et al., 2014)	9
Figure 2-6 Plan view of the Majiagou landslide (He et al., 2018)	10
Figure 2-7 Physical model of dip slope with thrust load (He et al., 2018).....	10
Figure 2-8 Failure state of soil of smooth wall.....	11
Figure 2-9 Loose sand deformation mobilised with passive wall translation: (a) vectorial displacement (scale=1·5); (b), (c) and (d) maximum shear strain (%)(Deng & Haigh, 2022).....	12
Figure 2-10 Dense sand deformation mobilised with passive wall translation: (a) vectorial displacement (scale=4); (b), (c) and (d) maximum shear strain (%)(Deng, 2020).....	12
Figure 2-11 Sand deformations mobilised with passive wall translation in 1g tests with dense sand (Gudehus & Nübel, 2004).....	13
Figure 2-12 Oblique-section of Area 4.1 with geological structures and slope geometry (modified from Pipatpongsa et al. (2022))	13
Figure 2-13 Photograph of landslide in Area 4.1 pit wall (modified from Pipatpongsa et al. (2022))	14
Figure 2-14 Photograph of landslide in Area 4.1 pit wall (modified Pipatpongsa et al. (2022))	14
Figure 2-15 Slope models inclined along bedding plane and subjected to surcharge loading	15
Figure 2-16 Mechanisms of compaction-induced passive-state earth pressure (Chen & Fang, 2008).....	15

Figure 2-17 Schematics of normal stress in lateral planes at top and bottom(Pipatpongsa et al., 2022)	16
Figure 2-18 Three-dimensional factor of safety with respect to reduction ratio, breadth and pressure head on bedding plane with variation breadth (modified Pipatpongsa et al. (2022))).....	18
Figure 2-19 Slide-to-flow landslide triggering mechanism(Take et al., 2004)	19
Figure 2-20 Slide-to-flow landslide triggering mechanism model(Take et al., 2004) ...	20
Figure 2-21 Theoretical stress path for infinite slope of varying inclinations.(Take et al., 2004).....	20
Figure 2-22 Transition from slide to flow in loose fill model (Take et al., 2004).....	21
Figure 2-23 Physical model of landslide triggering by seepage-induced(Take & Beddoe, 2014).....	22
Figure 2-24 (a) Flux boundary condition applied to slope model and (b) resulting pore-water pressure response(Take & Beddoe, 2014)	22
Figure 2-25 Vectors displacement during seepage induced events(Take & Beddoe, 2014).....	23
Figure 2-26 Structure of thesis and corresponding objectives	25
Figure 3-1 Schematic diagram of the experimental setup	27
Figure 3-2 1g experimental set up	27
Figure 3-3 Experimental procedure on 1g experiments	27
Figure 3-4 Uniform circular motion of solid circle	28
Figure 3-5 Principle of centrifuge modelling (Madabhushi, 2014).....	30
Figure 3-6 Error due to the radial gravity field (Madabhushi G., 2014).	31
Figure 3-7 Inclination of resultant acceleration.....	31
Figure 3-8 Beam centrifuge: (a) beam centrifuge in Kyoto University from a plan view ; (b) schematic view of the beam centrifuge.(Fang et al., 2023)	32
Figure 3-9 Schematic for designing the centrifuge landslide test.(modified from Fang et al. (2023))	33
Figure 3-10 Schematic diagram of triggering system of centrifuge model.....	35
Figure 3-11 Front of centrifuge model container	35
Figure 3-12 Top supply tank cover centrifuge model container	36
Figure 3-13 Slope frame of centrifuge model and bottom storage tank.....	36
Figure 3-14 Water supplementation	37
Figure 3-15 Drainage system of bottom storage tank and pump.....	37
Figure 3-16 Surcharge cart and loadcell.....	37
Figure 3-17 Control boxes of water supplementation and surcharge controller.	38

Figure 3-18 Pressure gauge	38
Figure 3-19 Laser sensor	39
Figure 3-20 Pore water pressure transducer	39
Figure 3-21 Accelerometer	40
Figure 3-22 Normal stress and shear stress of interface friction between Hiroshima sand and Teflon sheet.....	42
Figure 3-23 1g experimental preparation	43
Figure 3-24 Preparation of centrifugal model	44
Figure 3-25 Centrifugal model for dynamic experiment.....	44
Figure 3-26 Interrogation window of PIV analysis	46
Figure 3-27 Application of mask, ROI and PIV analysis.....	46
Figure 3-28 Diagram of PIV calculation	47
Figure 4-1 schematic of slope lying on bedding plane with lateral and toe support	48
Figure 4-2 Failure observation during experiment	50
Figure 4-3 Pixels on recorded video of dip slope	52
Figure 4-4 Application of mask and ROI	52
Figure 4-5 velocity vector field	53
Figure 4-6 Flowchart of statistical of slip line detection	53
Figure 4-7 Calculation process of coefficient of variation	55
Figure 4-8 Time of Minimum CV	56
Figure 4-9 The result of slip line detection from Minimum CV	57
Figure 4-10 Correlation between time of minimum CV, mean velocity (upper and lower) and standard deviation (upper and lower).....	58
Figure 4-11 Snapshots of the velocity vector distribution for test A1.....	59
Figure 4-12 Snapshots of the velocity vector distribution for test A2.....	59
Figure 4-13 Snapshots of the velocity vector distribution for test A3.....	59
Figure 4-14 Snapshots of the velocity vector distribution for test B.....	59
Figure 4-15 Snapshots of the velocity vector distribution for test C.....	60
Figure 4-16 Snapshots of the velocity vector distribution for test D.....	60
Figure 4-17 Snapshots of the velocity vector distribution for test E1	60
Figure 4-18 Snapshots of the velocity vector distribution for test E2	60
Figure 4-19 Snapshots of the velocity vector distribution for test F	61
Figure 4-20 Snapshots of the velocity vector distribution for test G1.....	61
Figure 4-21 Snapshots of the velocity vector distribution for test G2.....	61

Figure 4-22 Snapshots of the velocity vector distribution for test H.....	61
Figure 4-23 Contourplot of Minimum CV	63
Figure 4-24 Rankine's passive wedge and conceptual of dip slope slip line	64
Figure 4-25 selected area for inverse velocity analysis	65
Figure 4-26 Inverse velocity and bilinear regression based on moving area by minimum CV	66
Figure 4-27 Inverse velocity and bilinear regression based on selected 4 th area by minimum CV	67
Figure 4-28 Correlation between mobilized time of selected area and moving area	68
Figure 4-29 Correlation between time of failure based on inverse velocity and minimum CV	69
Figure 4-30 Correlation between time of failure based on selected area on inverse velocity and minimum CV	69
Figure 5-1 Idealization of slope lying on bedding plane with lateral confinements and toe support	72
Figure 5-2 Diagram of force acting on slope model.....	72
Figure 5-3 Diagram of acting force on separated slope and toe part.....	72
Figure 5-4 Groundwater seepage on slope lying on bedding plane	76
Figure 5-5 schematic diagram of triggering for centrifuge model	77
Figure 5-6 The instrumentation set up.....	78
Figure 5-7 Pointing position of laser sensors	79
Figure 5-8 Installation of instruments	80
Figure 5-9 Restricted mold and construction of slope model.....	80
Figure 5-10 A slope after compaction	80
Figure 5-11 Installation of laser sensor, surcharge and water supplementation.....	81
Figure 5-12 Installation of supply top tank	81
Figure 5-13 Experimental procedure	82
Figure 5-14 Experimental result of test 1	83
Figure 5-15 Slope failure observed after the testing of Test 1.	84
Figure 5-16 Experimental result of test 2	85
Figure 5-17 Slope failure observed after the testing of Test 2.	86
Figure 5-18 Experimental result of test 3	87
Figure 5-19 Slope failure observed after the testing of Test 3.	88
Figure 5-20 Experimental result of test 4	89
Figure 5-21 Slope failure observed after the testing of Test 4.	89

Figure 5-22 Experimental result of test 5	91
Figure 5-23 Slope failure observed after the testing of Test 5.	91
Figure 5-24 Experimental result of test 6	93
Figure 5-25 Slope failure observed after the testing of Test 6.	93
Figure 5-26 PIV analysis results of test 1	95
Figure 5-27 PIV analysis results of test 2	96
Figure 5-28 PIV analysis results of test 3	96
Figure 5-29 PIV analysis results of test 4	97
Figure 5-30 PIV analysis results of test 5	97
Figure 5-31 PIV analysis results of test 6	98
Figure 5-32 location of earth pressure gauge and pore water pressure sensors for calculating reduction ratio	98
Figure 5-33 Experimental reduction ratio	99
Figure 5-34 Factor of safety subjected pore-water pressure and applied force	101
Figure 5-35 Comparison between prediction and experimental factor of safety	102
Figure 5-36 Comparison of toe resistance and passive thrust limitation	103
Figure 5-37 Prediction of applied force	104
Figure 5-38 Correlation between predicted applied force and actual applied force	105
Figure 6-1 Idealization of mitigating counterweight (berm type 1) for slope 30° and toe 0°	107
Figure 6-2 Idealization of mitigating counterweight (berm type2) for slope 30° and toe 0°	107
Figure 6-3 Idealization of mitigating counterweight (various berm height)	107
Figure 6-4 Schematic diagram of triggering for centrifuge model	110
Figure 6-5 The instrumentation set up	112
Figure 6-6 Pointing position of laser sensors	112
Figure 6-7 Installation of instruments	113
Figure 6-8 Rotation of soil chamber and compaction	113
Figure 6-9 Construction of berm type 1 on slope 30° and toe 0°	114
Figure 6-10 Construction of berm type 2 on slope 30° and toe 0°	114
Figure 6-11 Construction of berm on slope 30° and toe 15°	114
Figure 6-12 Experimental procedure	115
Figure 6-13 Experimental result of test 1	117
Figure 6-14 Slope failure observed after the testing of Test1	117

Figure 6-15 Experimental result of test 2	119
Figure 6-16 Slope failure observed after the testing of Test2	119
Figure 6-17 Experimental result of test 3	121
Figure 6-18 Slope failure observed after the testing of Test3	121
Figure 6-19 Experimental result of test 4	123
Figure 6-20 Slope failure observed after the testing of Test4	123
Figure 6-21 Experimental result of test 5	125
Figure 6-22 Slope failure observed after the testing of Test5	125
Figure 6-23 location of earth pressure gauge and pore water pressure sensors for calculating reduction ratio	126
Figure 6-24 Experimental reduction ratio	127
Figure 6-25 PIV analysis results of test 1.....	128
Figure 6-26 PIV analysis results of test 2.....	128
Figure 6-27 PIV analysis results of test 3.....	129
Figure 6-28 PIV analysis results of test 4.....	129
Figure 6-29 PIV analysis results of test 5.....	130
Figure 6-30 Comparison between prediction and experimental factor of safety	131
Figure 6-31 Comparison of toe resistance and passive thrust limitation.....	132
Figure 6-32 The relationship between resistance and counterweight fill of slope angle 30degree with reverse toe angle 15degree.....	133
Figure 6-33 Prediction of applied force.....	134
Figure 6-34 Correlation between predicted applied force and actual applied force.....	135
Figure 7-1 Schematic diagram of triggering for centrifuge model.....	137
Figure 7-2 Experimental result of test 1	140
Figure 7-3 Slope failure observed after the testing of Test1	141
Figure 7-4 Experimental result of test 2	142
Figure 7-5 Slope failure observed after the testing of Test2	142
Figure 7-6 Experimental result of test 3	144
Figure 7-7 Slope failure observed after the testing of Test3	144
Figure 7-8 Experimental result of test 4	146
Figure 7-9 Slope failure observed after the testing of Test4	146
Figure 7-10 Experimental result of test 5	148
Figure 7-11 Slope failure observed after the testing of Test5	148
Figure 7-12 Experimental result of test 5	150

Figure 7-13 Slope failure observed after the testing of Test5	150
Figure 7-14 location of earth pressure gauge and pore water pressure sensors for calculating reduction ratio	151
Figure 7-15 Experimental reduction ratio	152
Figure 7-16 Adjusted groundwater of dip slope on permeable bedding plane.....	153
Figure 7-17 Comparison between prediction and experimental factor of safety	154
Figure 7-18 Comparison of toe resistance and passive thrust limitation.....	156
Figure 7-19 Prediction of applied force.....	157
Figure 7-20 Correlation between predicted applied force and actual applied force.....	158
Figure 8-1 Laterally confined slope lying on bedding plane and toe support.	159
Figure 8-2 The resistance force against slope sliding.....	160
Figure 8-3 Schematic diagram of the experimental setup	163
Figure 8-4 Slope model with instrumentations.....	163
Figure 8-5 Soil chamber and model (model scale).....	163
Figure 8-6 Datum input wave (prototype scale)	164
Figure 8-7 Captured image from video high-speed camera	166
Figure 8-8 Slope displacement versus time (prototype scale).	166
Figure 8-9 Recorded earth pressure during the application of shaking (prototype scale).	167
Figure 8-10 Reduction factor during fourth shaking	168
Figure 8-11 Comparison between theoretical prediction and experimental results	168
Figure 8-12 Comparison between theoretical prediction and experimental results	169

List of Tables

Table 3-1 Scaling law of centrifuge modelling	30
Table 3-2 Basic properties of Silica sand No. used in 1g test (Fang, 2019).....	40
Table 3-3 Summary of interface strength parameters	41
Table 3-4 Basic properties of Hiroshima sand	41
Table 3-5 Summary of interface shear strength for Hiroshima sand.....	42
Table 3-6 Interface shear strength between Hiroshima sand and Porous stone	42
Table 3-7 Basic properties of Edosaki sand used in a slope model.....	43
Table 4-1 Maximum surcharge and pressure in laterally confined slope models subject to a toe support	49
Table 4-2 Summation of statistical calculation	64
Table 4-3 Summation of mobilize time and failure time.....	68
Table 5-1 Basic properties of Hiroshima sand	78
Table 5-2 Configuration of slope model lying bedding plane with lateral, and toe supports.....	79
Table 5-3 The experimental results of dip slope under the influence of groundwater seepage.....	94
Table 6-1 Basic properties of Hiroshima sand	111
Table 6-2 Geometries of slope model lying bedding plane with lateral, toe supports and various berm types.....	111
Table 7-1 Configuration of slope model lying bedding plane with lateral, toe supports and berm.	138
Table 8-1 Material properties of Edosaki sand and Physical properties (Aroonwattanaskul, 2020)	164
Table 8-2 Fourth instances of seismic waves (prototype scale)	164
Table 8-3 Reduction ratio during shaking (prototype scale).	168

Chapter 1 Introduction

Background

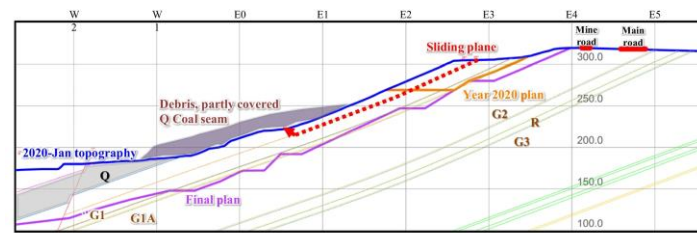
Slopes lying on bedding planes can be widely found in both natural and human-made environments. A slope lying on bedding, often referred to as a dip slope, is a geological formation where the slope surface parallels or closely follows the orientation of underlying sedimentary rock layers or bedding planes. This type of slope is common in areas with sedimentary rock formations where layers of rock have been tilted by tectonic forces (Wyllie & Mah, 2004). Defined by their weak planes dipping in the same direction as the slope face, dip slopes have caused numerous disasters in recent decades due to their propensity for failure (Fisher, 2009; Lo & Weng, 2017; Stead & Eberhardt, 1997). Translational sliding is the typical type of motion associated with the failure of dip slopes (Varnes, 1978). This type of failure often has severe consequences due to its suddenness and rapidity. Understanding the behavior of dip slopes is crucial, particularly in the context of slope stability, a key concern in geotechnical engineering.

On April 25, 2010, a landslide occurred on Freeway No. 3, as illustrated in Figure 1-1. This event involved over 200,000 cubic meters of dirt and rocks crashing onto the roadway, destroying an overpass, resulting in four fatalities, and blocking the freeway between Keelung and Taipei. Detailed information regarding the landslide on Freeway No. 3 can be found in previous research (Wang et al., 2013; Yeh et al., 2021). The slope geometry on Freeway No. 3 consisted of a dip slope made up of shale and sandstone. During construction, the toe slope was excavated, and 572 rock anchors were installed to support the slope, with the rock anchor installation finished in 1998. A field investigation following the slope failure in 2010 revealed that 58 rock anchors remained in place after the landslide. In this case study, the loss of rock anchors' effectiveness led to decreased slope stability, resulting in a sudden landslide due to the removal of the toe slope. Therefore, the stability of the toe slope played a crucial role in maintaining overall slope stability. To identify sliding mechanism and to provide early warnings of dip slope failure are required (Lo et al., 2016; Wang et al., 2013).



Figure 1-1 (a) Photo displaying Freeway no.3 landslide in Taiwan from NASC Taiwan (2010). (b) Photo displaying disaster relief in progress from NASC Taiwan (2010), The picture in the upper right corner of (b) is a close-up view of the pre-disaster slope, which is marked as a yellow dash circle in (a) and (b) (Yeh et al., 2021)

Another case study is the Mae Moh open-pit mine, operated by the Electricity Generating Authority of Thailand (EGAT), which is the largest open-pit mine in Thailand. The mine features a slope situated on a bedding plane, also known as a dip slope (Corsiri & Crouch, 1985; Evans & Jitapankul, 1989; Tandicul et al., 1986). Over its 50-year operation, the mine has experienced landslides along these planes, which can cause significant damage to people and property downslope. Figure 1-2(a) and (b) illustrate the slope geometry and a resulting landslide, highlighting the large mass displacement. Instrument-detected displacements (Figure 1-3) confirm slope failure, particularly during the rainy season, while the slope remains stable during dry conditions. To identify sliding mechanism of dip slope with the rainfall infiltration are needed.

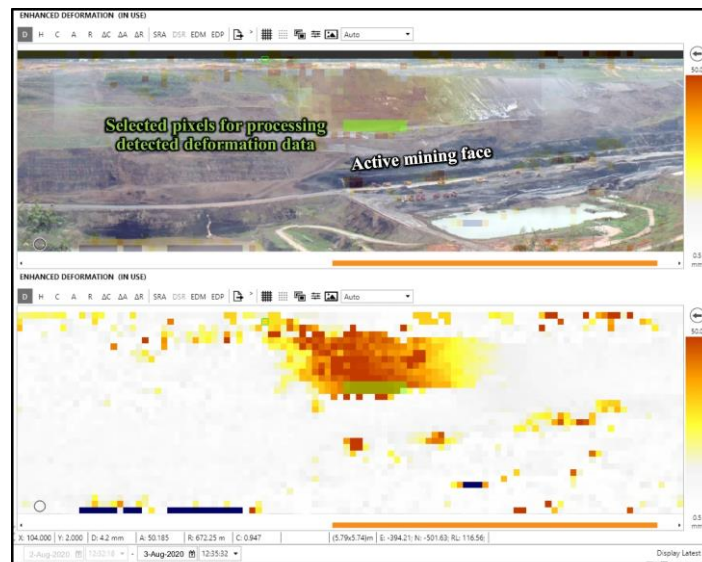


(a) Cross section at S11

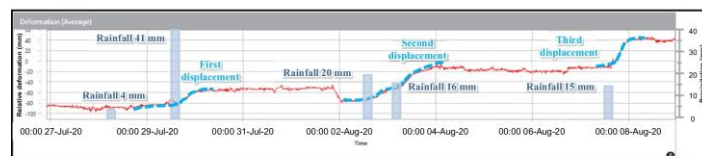


(b) Photograph of low wall failure

Figure 1-2 Photograph of low wall failure (Pipatpongsa et al., 2023)



(a) Low wall deformation heat map



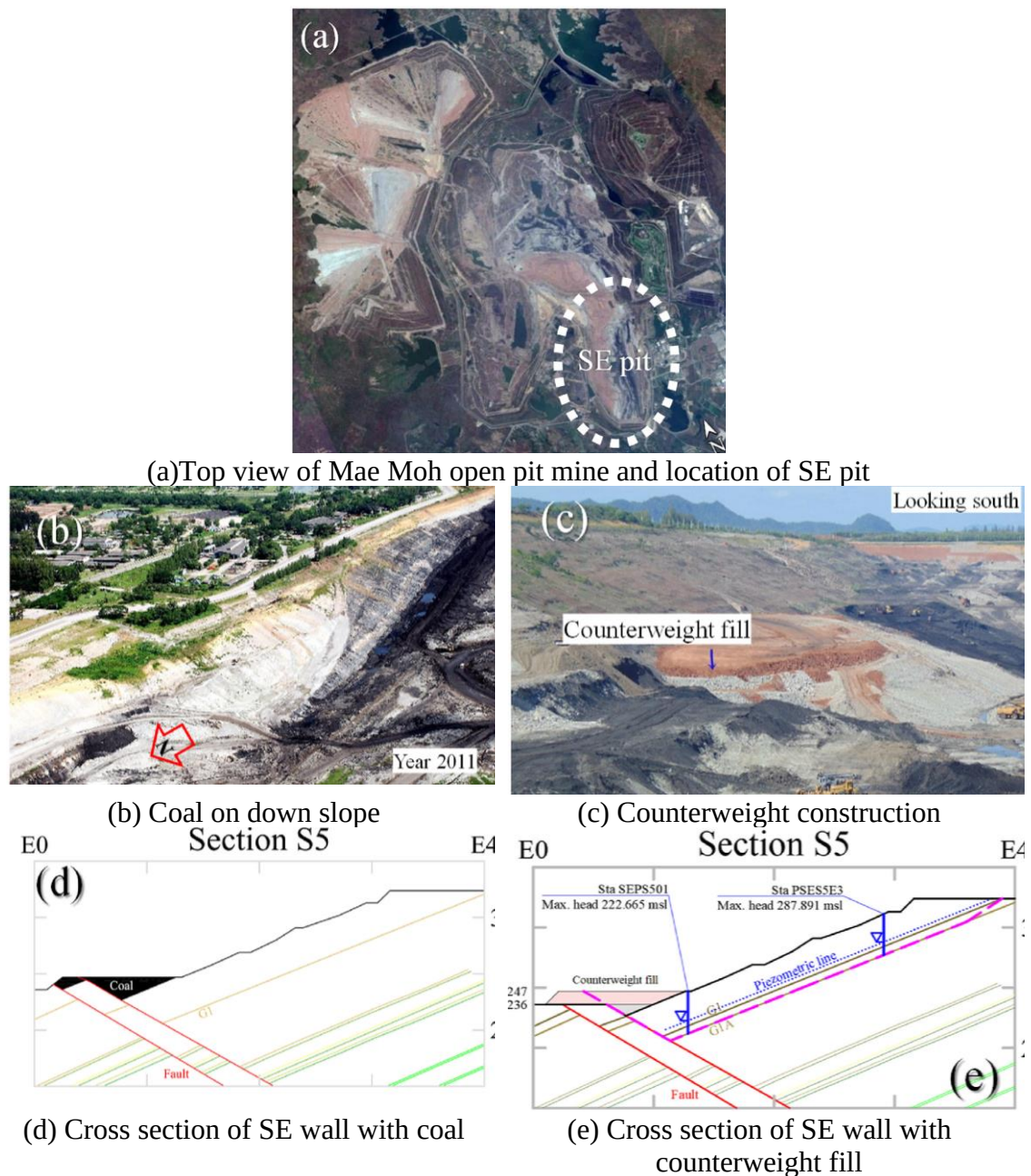
(b) Relative displacements and precipitation from July 27th to August 8th, 2020



(c) Relative displacements with no rainfall throughout dry season in 2020

Figure 1-3 Monitoring of surface slope displacement (Pipatpongsa et al., 2023)

To enhance stability and prevent sliding, a counterweight at the toe of the slope is used in Mae Moh open pit mine. Initially, coal served as a counterweight at the SE wall of the mine, but it was later used for electricity generation, necessitating a replacement to maintain slope stability. Figure 1-4(b) and (e) show the counterweight's effect on slope stability, with the profile before and after counterweight construction depicted in Figure 1-4(e) and (f). However, the lifted-up counterweight occurred later and led to slope sliding because the counterweight was insufficient to maintain slope stability. Therefore, it is necessary to study the optimum counterweight fill to ensure adequate slope stability.



Regarding the following case study of a dip slope landslide, the failure mechanisms of dip slopes have been studied through site investigation and numerical analysis (Chai, 2023; Havaej et al., 2014; Kvpil & Clews, 1979; Ning et al., 2011; Stead & Eberhardt, 1997; Stead et al., 2006; Supandi, 2021). The main failure mechanisms are divided into two types: bi-linear sliding and ploughing. Bi-linear sliding occurs when the dip slope slides along the bedding planes due to the loss of stability at the toe (sliding at toe). Ploughing is another type of dip slope failure mechanism where the slope slides along discontinuities while the toe remains stable. Based on these two failure modes, a physical model was constructed to study the effect of discontinuity joints on dip slope stability (Alejano et al., 2011; Lo & Weng, 2017). The findings reveal that the angle of the discontinuity joint plays an important role in determining the failure mechanism. However, further investigation is needed to explore a variety of discontinuity joints and failure mechanisms. To investigate the stability of dip slopes, numerical analysis and the limit equilibrium method (LEM) are commonly used to assess slope stability (Alejano & Sánchez Juncal, 2010; Stead & Eberhardt, 1997). However, to assess the impact of rainfall infiltration on dip slope stability, both physical modeling and slope stability analysis considering the effects of rainfall need to be studied.

Based on the background described above, this thesis aims to address the stability challenges associated with dip slopes through a comprehensive investigation. This research will design physical models using both 1g experiments and centrifuge experiments to achieve an understanding of the failure mechanisms of dip slopes subjected to various factors, particularly groundwater seepage. Building on previous research, the analysis of dip slope stability will be enhanced to better assess past and future dip landslides. Furthermore, the stability analysis will be validated through the mentioned experiments.

Chapter 2 Literature Review

In this chapter, the scope of this thesis is clarified through comprehensive literature reviews on dip slope. Firstly, in **Section 2.1**, the failure mechanisms on dip slope subject to toe orientation are introduced to demonstrate the method and emphasize the critical failure modes needed for study. **Section 2.2** introduces the physical modeling of a slope with thrust load application. The thrust load helps to simulate large boundary conditions, which is essential due to the limitations of laboratory apparatus. **Section 2.3** introduces site investigation and physical modeling with lateral confinement to analyze dip slope stability under lateral confinement. By considering the effect of lateral confinement on slope stability, the analysis is enhanced from two-dimensional (2D) to three-dimensional (3D). **Section 2.4** reviews the effects of groundwater infiltration through previous research to showcase physical modeling of dip slope subjected to groundwater infiltration. Finally, **Section 2.5** summarizes the topics addressed in this thesis based on the reviews from sections 2.1 to 2.4 and outlines the chapter structures of the thesis.

2.1 Failure mechanisms of slope lying on bedding plane

A slope lying on bedding, often referred to as a dip slope, is a geological formation where the slope surface parallels or closely follows the orientation of underlying sedimentary rock layers or bedding planes. This type of slope is common in areas with sedimentary rock formations where layers of rock have been tilted by tectonic forces (Wyllie & Mah, 2004). The geological configuration of dip slopes inherently increases the risk of landslides, especially when external factors like groundwater infiltration or human activities destabilize the slope (Goodman, 1991; Zhang et al., 2011). Surface coal mining, especially in mountainous regions, frequently results in the creation of extensive footwall slopes that run parallel to the dip of the strata. Footwall slope can occur in a different failure mode (Stead & Eberhardt, 1997)

- Bilinear stab

One mode failure mode is bilinear slab failure (see Figure 2-1), In the case of a high slope with sufficient driving force, failure may occur either by sliding along an upslope crossover surface and/or by shear failure at the toe of the slope or toe sliding. Dip slope can be separated into two parts, active and passive zone, active

zone belongs to slope due to weight are pass to toe of slope, then toe part is passive due to receive weight from upper slope. Prandtl wedge or transition zone is zone that transmits the driving force of upper slope to toe block. It can easily be found in mining engineering (Hawley et al., 1986)

- Ploughing failure

Another type of failure mechanism of dip slope is ploughing stab failure. It is a rare case in practical (Calder & Blackwell, 1980). The slope slides along a primary discontinuity and a joint, potentially causing the toe to lift or rotate, as showed in Figure 2-2.

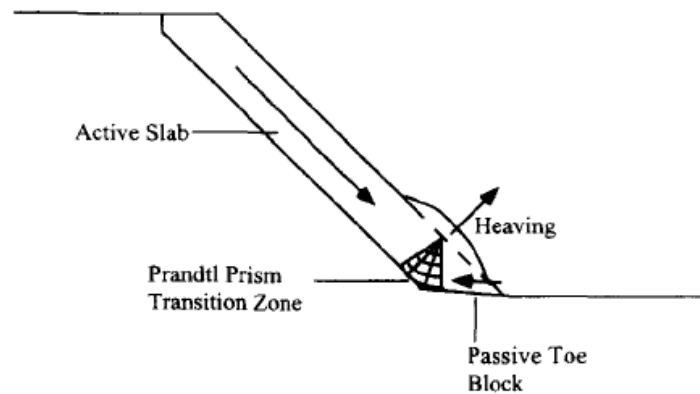


Figure 2-1 Bilinear footwall failure and location of transition zone(Stead & Eberhardt, 1997)

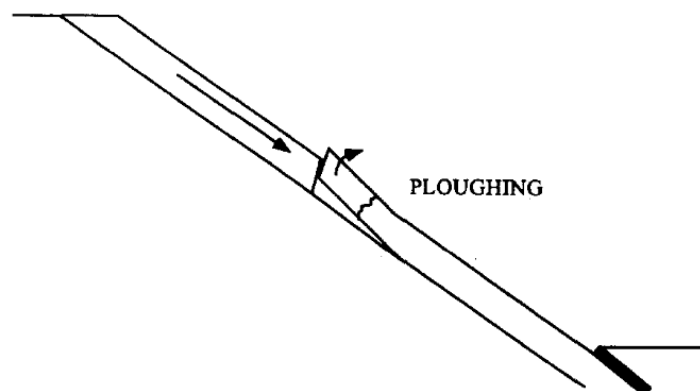


Figure 2-2 Mechanism of ploughing stab failure in foot wall slope(Stead & Eberhardt, 1997)

The limit equilibrium method was validated as shown in Figure 2-3 (Alejano et al., 2011) In the analysis of bilinear slab stability, which involves active and passive blocks in two different failure modes, the forces and factor of safety can be expressed. The slope was separated into two parts, which were triangular(passive) and rectangular(active). This method helps to accurately assess the stability of slopes by considering the interaction between different failure blocks and calculating the corresponding safety factors.

$$N_A = \frac{W_1 [\sin \theta - \cos \theta \tan \phi_1]}{1 - \tan \phi_A \tan \phi_1} \quad (2-1)$$

$$FOS = \frac{[W_1 \cos \theta_2 + N_A [\sin(\theta - \theta_2) + \cos(\theta - \theta_2) \tan \phi_A]] \tan \phi_2}{W_1 \sin \theta_2 + N_A [\cos(\theta - \theta_2) - \sin(\theta - \theta_2) \tan \phi_A]} \quad (2-2)$$

Where,

W_1, W_2 : the weights of the upper (or active) and lower (or passive) slabs

θ : the dip of the slope and the bedding planes

θ_2 : the dip of the toe joint

ϕ_1 : the friction angle of the bedding plane

ϕ_2 : the friction angle of the toe joint

Similarly, the force and factor of safety in the ploughing slab failure are calculated as follow (see Figure 2-4).

$$N_A = \frac{W_1 [\sin \theta - \cos \theta \tan \phi_1]}{\sin(\theta_A - \theta) [1 - \tan \phi_A \tan \phi_1] + \cos(\theta_A - \theta) (\tan \phi_A + \tan \phi_1)} \quad (2-3)$$

$$FOS = \frac{[N_A [\sin(\theta_A - \theta) + \cos(\theta_A - \theta) \tan \phi_A] + W_2 \sin \theta] \tan \phi_2}{N_A [\cos(\theta_A - \theta) - \sin(\theta_A - \theta) \tan \phi_A] - W_2 \cos \theta} \quad (2-4)$$

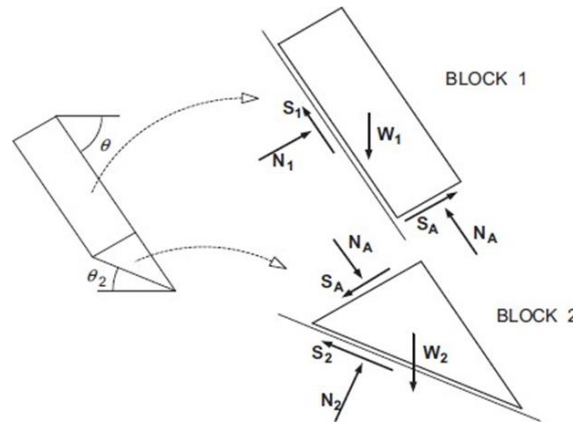


Figure 2-3 Geometric examination of bilinear slab failure(Alejano et al., 2011)

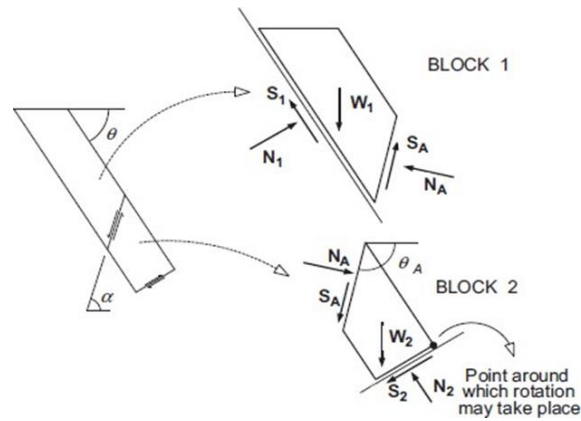


Figure 2-4 Geometric examination of ploughing slab failure (Alejano et al., 2011)

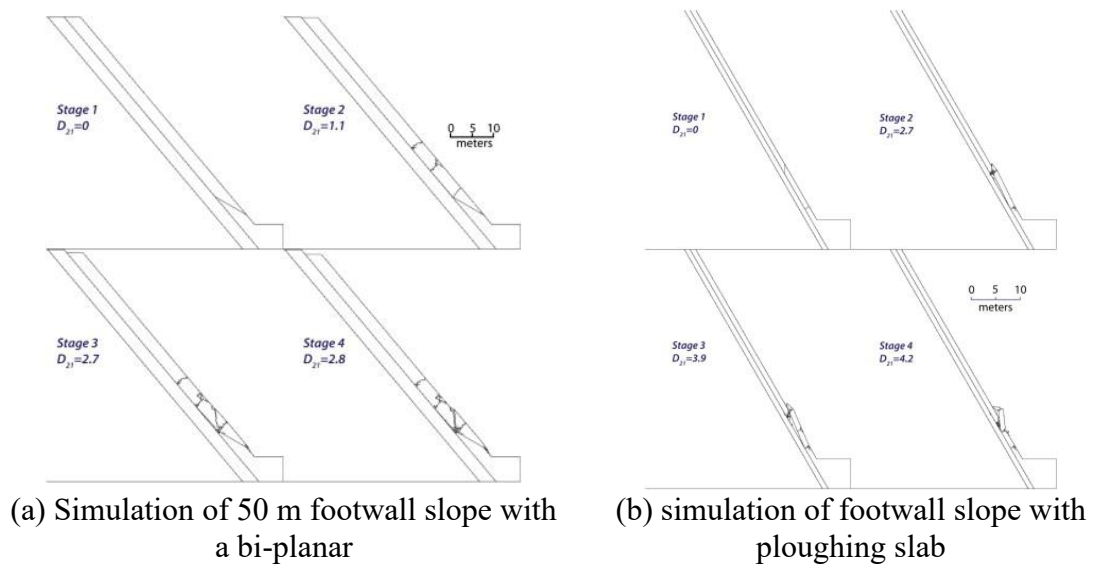


Figure 2-5 Simulation results of footwall analysis (Havaej et al., 2014)

The failure mechanism of footwall slopes was confirmed through numerical modeling by (Havaej et al., 2014). Their analysis highlights the significant role of toe joints in these failures (see Figure 2-5). If the joint at the toe declines downward along the slope, bilinear stability can occur. Conversely, if the joint is perpendicular to the slope, the failure mechanism resembles ploughing. These insights are crucial for understanding and managing slope stability in surface coal mining operations.

2.2 Physical modelling of slope lying bedding plane triggered by thrust load

A slope model test, wherein a dip slope is triggered by a thrust load, is widely used to simulate a driving force load. This method provides a simple way to represent the upper part of the sliding mass or the continuous accumulation of colluvial deposits at the trailing edge of the landslide (He et al., 2018; Li et al., 2016). Because prototype slopes at sites can be very extensive, constructing long slope models under laboratory conditions is often impractical. Therefore, a loading system was utilized in physical slope model tests. These simulations have been frequently used in physical slope model studies (He et al., 2018; Hu et al., 2019; Li et al., 2016; Lin & Lo, 2018; Liu et al., 2020; Lo & Weng, 2017). He et al. (2018) demonstrated using a thrust load to simulate the upper landslide mass. Figure 2-6 shows a plan view of the site investigation of the Majaigou landslide. Creating a physical model and triggering it to failure is challenging because the slope model requires a substantial load at the model scale. Therefore, a hydraulic ram and thrust plate were used to create a large mass boundary as a thrust load (see Figure 2-7).

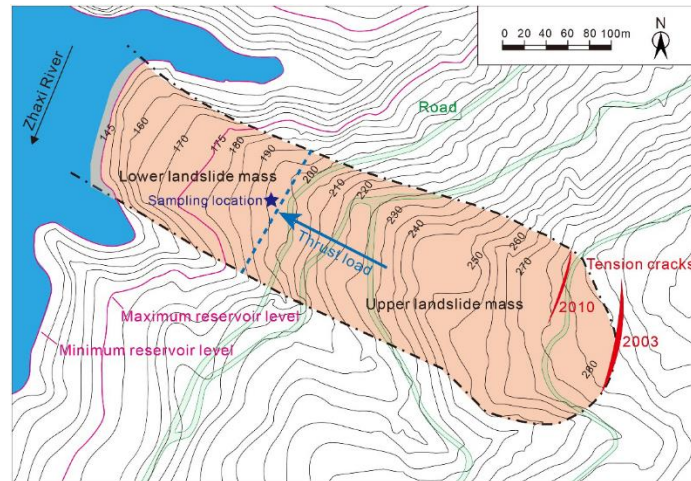


Figure 2-6 Plan view of the Majaigou landslide (He et al., 2018)

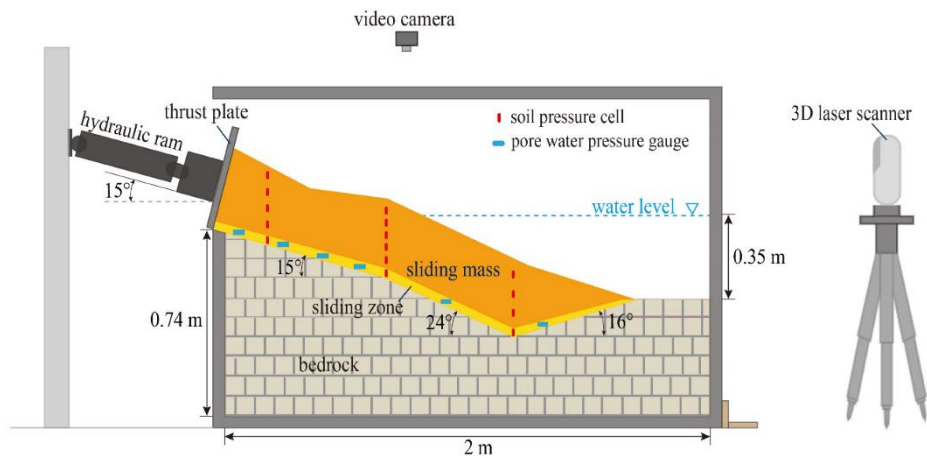


Figure 2-7 Physical model of dip slope with thrust load (He et al., 2018)

To simulate large load boundaries, it is essential to study soil deformation behind wall translation. According to the lateral earth pressure theory, the pressure exerted by soils on retaining walls changes horizontally with wall movement, either inward or outward. When a thrust load is applied, causing the wall to move into the soil, shear stress resists the force pushing the wall. This resistance results in increased earth pressures, known as passive earth pressure. The concept of the earth pressure coefficient, K , as defined in Eq. 2-1, is incorporated into earth pressure analysis to quantify these variations.

$$K = \frac{\sigma'_h}{\sigma'_v} \quad (2-5)$$

Rankine earth pressure the theory (Rankine, 1857)

$$K_a = \tan^2(45^\circ - \frac{\phi}{2}) \quad (2-6)$$

$$K_p = \tan^2(45^\circ + \frac{\phi}{2}) \quad (2-7)$$

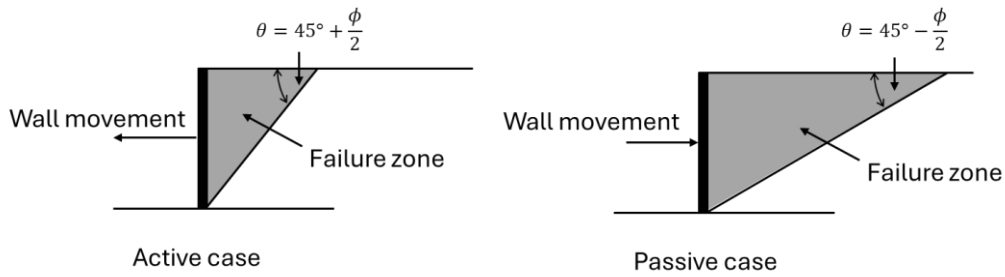


Figure 2-8 Failure state of soil of smooth wall

Deng and Haigh (2022) conducted research on the upward translation of a wall relative to soil in both loose and dense sand conditions. Centrifuge experiments were performed with the wall moving towards the sand under both conditions. GeoPIV was applied in this research to visualize soil deformation during wall movements. The results for loose sand conditions are presented in Figure 2-9, showing soil deformation in terms of displacement vectors and color plots of displacement fields. The results indicate that displacement vectors vary with wall height, with the top of the wall exhibiting greater displacement vectors than the bottom. Similar results were observed for dense sand conditions, as shown in Figure 2-10, where large displacements occurred near the wall surface.

A comparison was made with conventional 1g tests by Gudehus and Nübel (2004), as illustrated in Figure 2-11. Passive wall translation generated a rupture surface inclined at approximately $45^\circ - (1/2)\phi$ to the horizontal. That result matched Rankine passive wall movement theory.

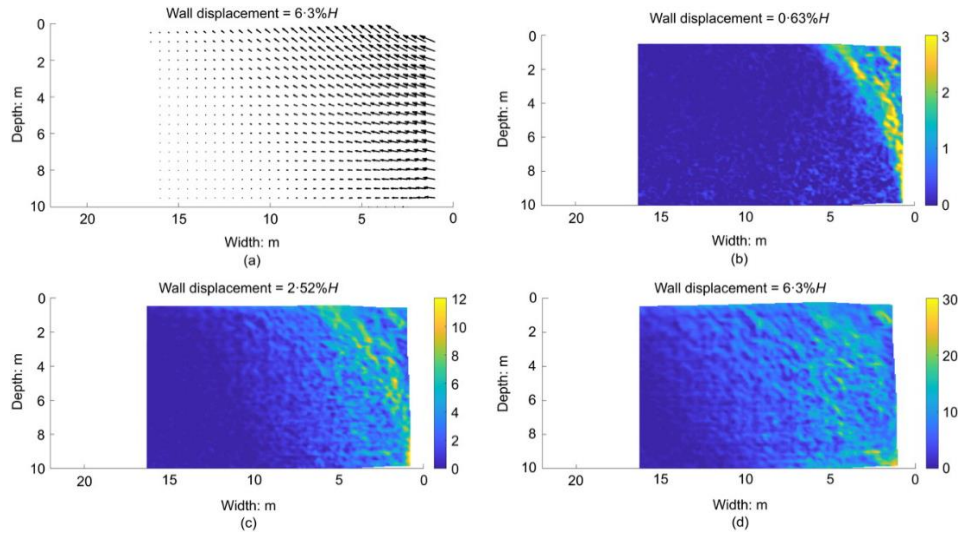


Figure 2-9 Loose sand deformation mobilised with passive wall translation: (a) vectorial displacement (scale=1.5); (b), (c) and (d) maximum shear strain (%) (Deng & Haigh, 2022)

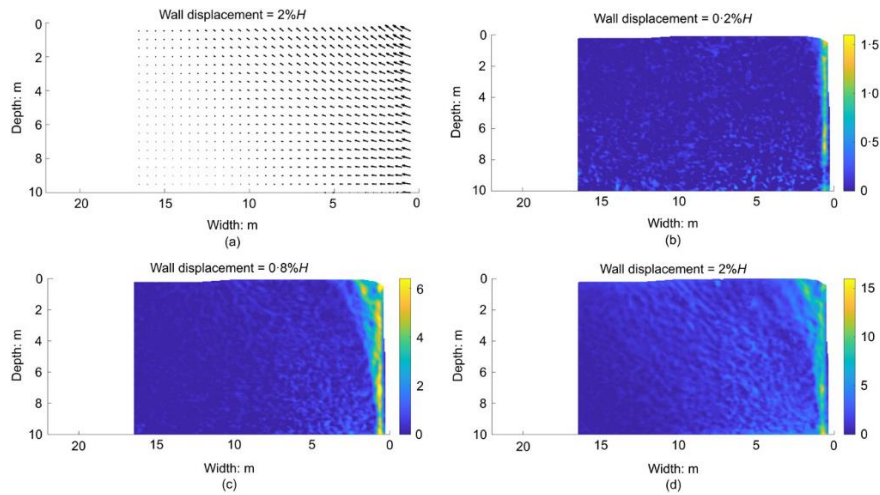


Figure 2-10 Dense sand deformation mobilised with passive wall translation: (a) vectorial displacement (scale=4); (b), (c) and (d) maximum shear strain (%) (Deng, 2020)

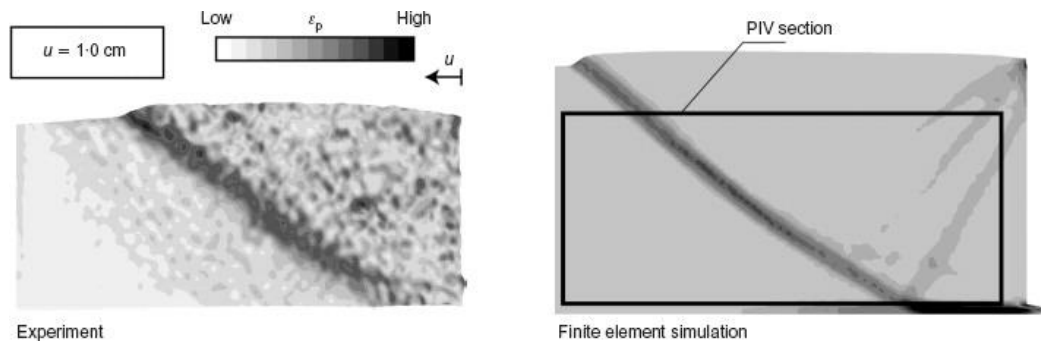


Figure 2-11 Sand deformations mobilised with passive wall translation in 1g tests with dense sand (Gudehus & Nübel, 2004)

2.3 Lateral confinement on slope lying on bedding plane

In year 2012 to 2017, the northeast pit wall of the mine was experimented as a largescale undercutting slope, called “Area 4.1”. Figure 2-12 illustrates the oblique-section and pit contour, which illustrates the slope geometry of Area 4.1. The slope of pit wall overlaid on G1, which is a thin weak clay layer, there was approximately 250 meters in length and thickness is around 10 to 20 meters. The average angle of inclination of the Bedding Plane of G1 layer is 16° (from horizontal direction). The interface friction angle between G1 and grey claystone rock mass was recorded as 12° to 14° . The slope wall collapsed 5 year later, During the rainy season on October 3rd in year 2018, area 4.1 a central part of undercut pit wall noticeably failed on the daylighted G1 interface. The slope interface friction angle was reported as low angle. Moreover, slope wall appeared width of still daylighted approximately 190 to 200 meters, daylighted G1 interface is illustrated as green line in Figure 2-14. On the two sides of the landslide, vertical shear planes, also known as flanks, provide evidence that the rock mass failed. Two sides of lateral confinement can be obviously observed in Figure 2-13. Therefore, this example of dip slope failure along bedding plane with 2 side lateral confinements.

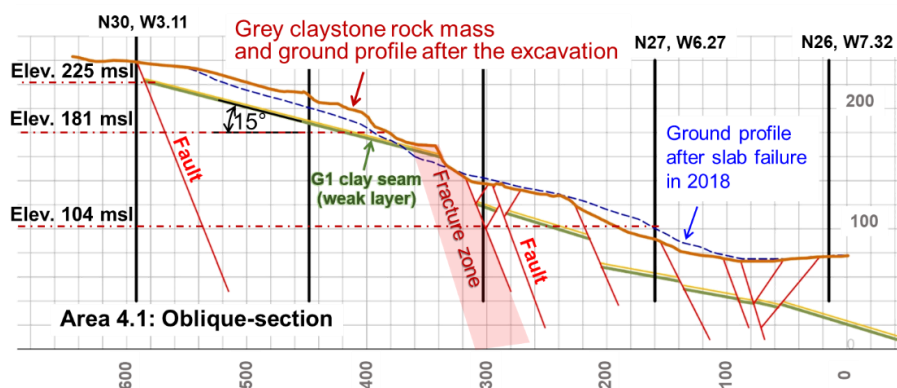


Figure 2-12 Oblique-section of Area 4.1 with geological structures and slope geometry (modified from Pipatpongsa et al. (2022))

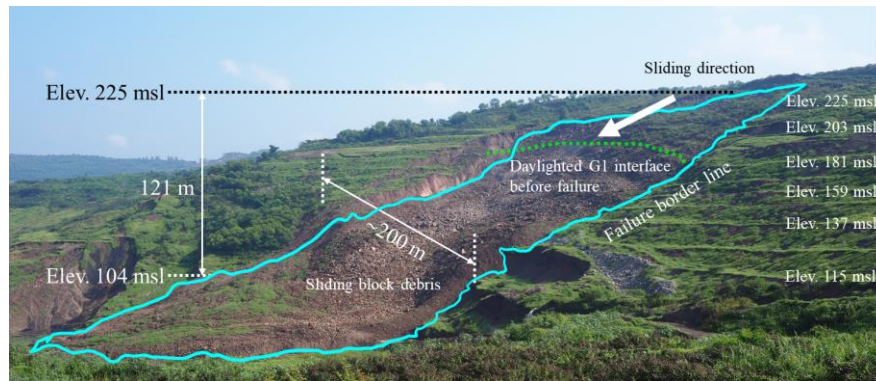


Figure 2-13 Photograph of landslide in Area 4.1 pit wall (modified from Pipatpongsa et al. (2022))

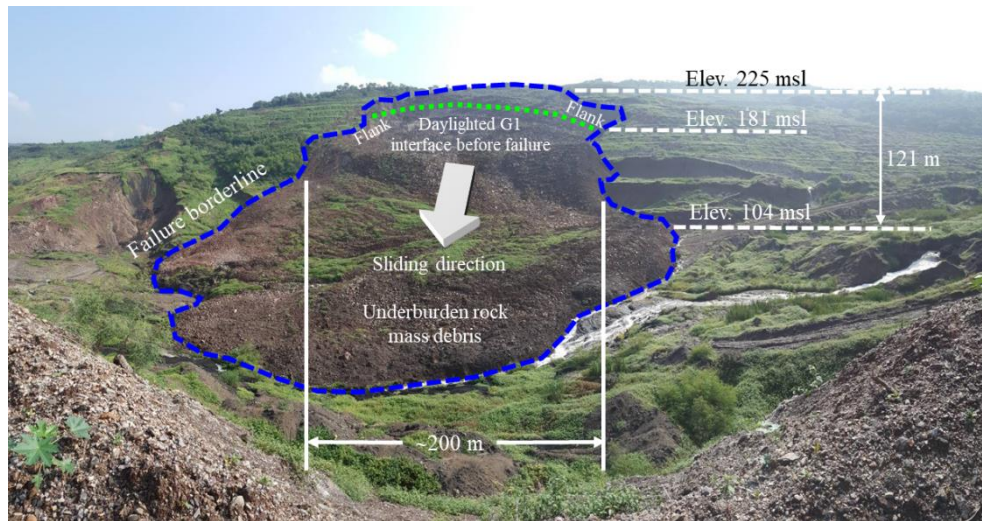


Figure 2-14 Photograph of landslide in Area 4.1 pit wall (modified Pipatpongsa et al. (2022))

Pipatpongsa et al. (2022) proposed a physical model of a slope lying on a bedding plane with lateral confinements. This 1g physical model was designed to address slope failure under site-specific conditions. As shown in Figure 2-15, the slope model was constructed along a bedding plane with two lateral supports. The model was triggered by a surcharge cart to simulate a large moving boundary. Slope stability can be calculated by dividing the resistance force by the driving force. In this model, two types of resistance are present: bedding resistance and lateral resistance from the two sides. The driving force is the combination of the surcharge weight and the slope's self-weight.

The method to calculate bedding resistance involves the normal force and the Mohr-Coulomb criteria, which depend on the interface properties between the soil material and the bedding material. Similarly, lateral resistance can be calculated using the Mohr-Coulomb criteria. However, the normal force acting on the lateral supports needs to be explained by lateral earth pressure according to Rankine's earth pressure theory.

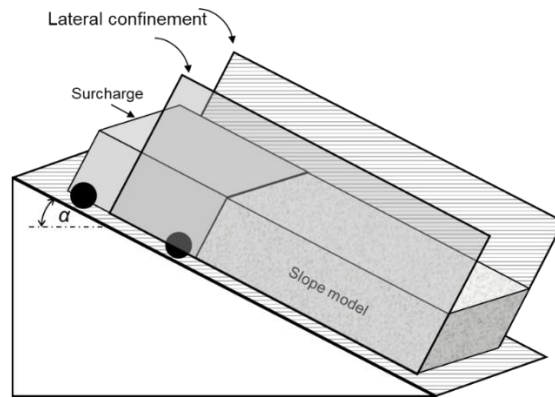
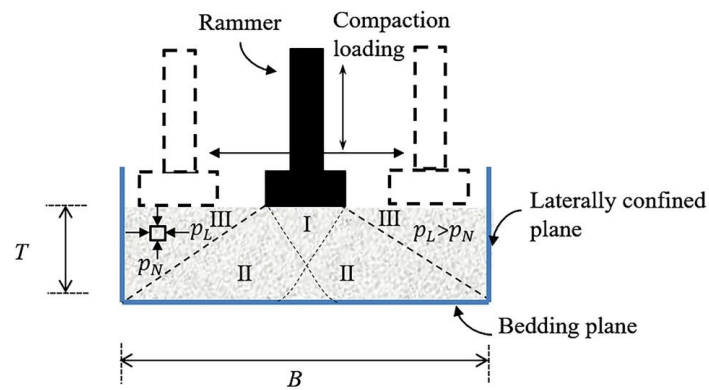


Figure 2-15 Slope models inclined along bedding plane and subjected to surcharge loading

Chen and Fang (2008) observed that earth pressure on a rigid retaining wall changes as a result of compacting cohesionless backfill. Their findings show that a passive state zone progressively forms in Zone III during the compaction process. This development is depicted in Figure 2-16, where the use of a rammer to compact the soil generates passive earth pressure on the retaining wall or lateral confining plane.



I: Elastic zone; II: Radial shear zone; III: Passive state zone

Figure 2-16 Mechanisms of compaction-induced passive-state earth pressure (Chen & Fang, 2008)

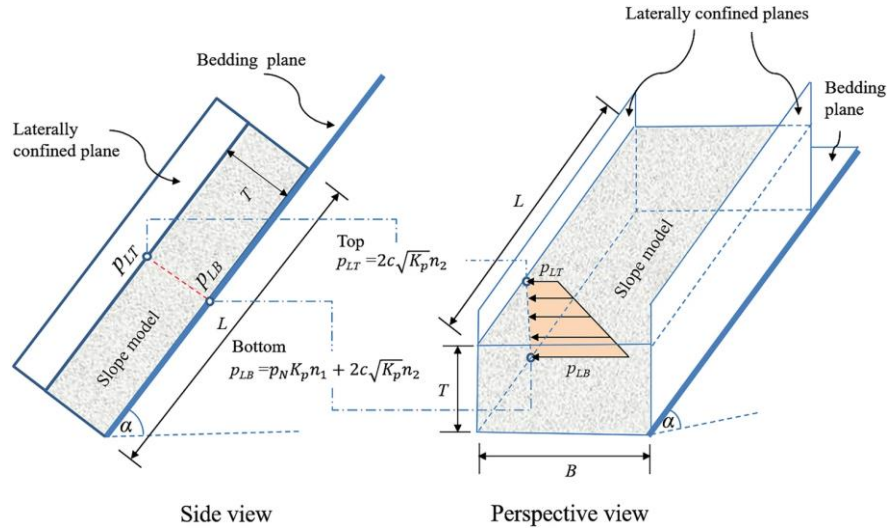


Figure 2-17 Schematics of normal stress in lateral planes at top and bottom(Pipatpongsa et al., 2022)

This study builds upon the framework proposed by Chen and Fang (2008), extending it from dry soil to humid soil conditions by incorporating the concept of apparent cohesion into Rankine's passive pressure. Additionally, it assumes that the pre-compression induced by the passive earth pressure persists throughout the model tests, provided that the lateral confinement remains intact, and the compacted soil remains stable. When the lateral earth pressure reaches its maximum capacity, the average Rankine's earth pressure, denoted as P_Y , acting on the laterally confined plane, is determined by Eq. (2-6), where the coefficient of earth pressure, K , is defined by Eq. (2-7). The calculation of horizontal earth pressure can be derived from vertical earth pressure, P_N , using Eq. (2-5) based on Rankine's earth pressure theory.

$$P_N = \frac{F_N}{BL} = \gamma T \cos \alpha \quad (2-5)$$

$$P_Y = \frac{L \int_0^T (K(r_d) \gamma t \cos \alpha + 2c \sqrt{K(r_d)}) dt}{TL} = \frac{1}{2} P_N K(r_d) + 2c \sqrt{K(r_d)} \quad (2-6)$$

$$K(r_d) = \tan^2 \left(\frac{\pi}{4} + \frac{r_d \tan^{-1} \phi}{2} \right) \quad (2-7)$$

Reduction ratio

The reduction ratio is a vital parameter in determining both active and passive earth pressure. Additionally, it serves as a basis for calculating lateral resistance based on earth pressure acting on lateral support. Following Rankine's earth pressure theory, the effective horizontal stress (σ'_h) is affected by the vertical stress (σ'_v) and is constrained

by the active and passive earth pressures, as expressed in Eqs. (2-8) and (2-9), respectively. However, in the case of a dip slope triggered by surcharge loading and compressing the slope model, passive earth pressure is expected to occur.

$$\sigma'_h = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right) \sigma'_v - 2c \tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right) \quad (2-8)$$

$$\sigma'_h = \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \sigma'_v + 2c \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \quad (2-9)$$

The transition of σ_h from active earth pressure to passive earth pressure can typically be linked to the shear strength reduction ratio r_d and the coefficient of lateral earth pressure K through Eqs. (2-10) and (2-11).

$$\sigma'_h = K \sigma'_v + 2r_d c \sqrt{K} \quad (2-10)$$

where

$$\sqrt{K} = \tan\left(\frac{\pi}{4} + \frac{\tan^{-1}(r_d \tan \phi)}{2}\right) \quad (2-11)$$

r_d span a range from -1 to 1 . Accordingly, the effective horizontal stress is in the active state when $r_d < 0$, isotropic when $r_d = 0$, and in the passive state when $r_d > 0$. With respect to Eq. (2-11), r_d can be expressed by the following equation:

$$r_d = \frac{1}{2 \tan \phi} \frac{K - 1}{\sqrt{K}} \quad (2-12)$$

By substituting Eq. (2-12) into Eq. (2-10), K can be determined as shown in Eq. (5-6).

$$K = \frac{\sigma'_h \tan \phi + c}{\sigma'_v \tan \phi + c} \quad (2-13)$$

To eliminate K from Eq. (2-12), we can substitute Eq. (2-11) into Eq. (2-12). Consequently, r_d can be computed using Equation (2-13), utilizing material properties c and ϕ , along with σ'_v and σ'_h measured at an identical depth.

$$r_d = \frac{\sigma'_h - \sigma'_v}{\sqrt{(\sigma'_v \tan \phi + c)(\sigma'_h \tan \phi + c)}} \quad (2-13)$$

To prevent confusion with the positively defined shear strength reduction ratio used in the shear strength reduction technique, in this study, r_d is referred to as the reduction ratio, especially since r_d can be negative in an active state.

The lateral shear resistance R_L is defined using Mohr's coulomb criteria. Here, ϕ_s represents the interface friction angle, while c_s denotes the adhesion between the sand and the lateral supports.

$$R_L = (P_L \tan \phi_s + c_s)LT \quad (2-14)$$

Thus, the inclusion of lateral shear resistance R_L enhances the ability to calculate the factor of safety for slopes lying on bedding planes, advancing from two-dimensional to three-dimensional analysis. This improvement is applicable in both site investigations and laboratory testing, providing a more comprehensive understanding of slope stability.

As indicated by the parametric study conducted by Pipatpongsa et al. (2022)(see Figure 2-18), varying the breadth (B) while maintaining the reduction ratio (r_d) constant at 0.71 showed that the lateral resistance (R_L) remained stable due to the fixed thickness (T) of 18.8 m. However, the bedding resistance (R_B) increased as a result of the enlarged slope area. Consequently, the factor of safety for the dip slope decreased as the weight of the slope increased, demonstrating that variations in B have no significant impact on lateral resistance (R_L).

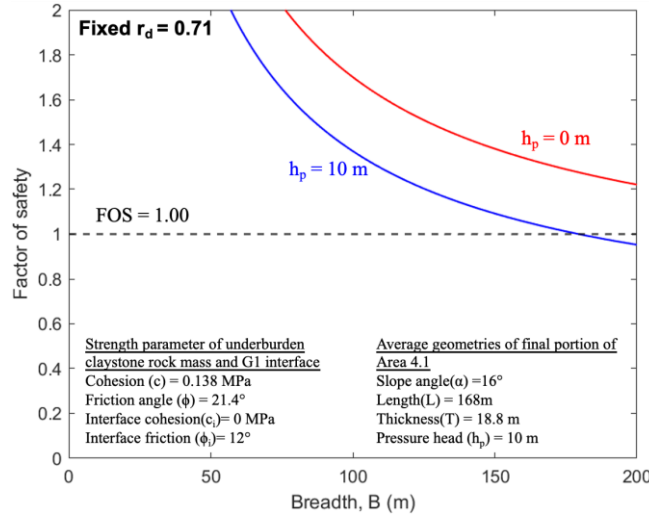


Figure 2-18 Three-dimensional factor of safety with respect to reduction ratio, breadth and pressure head on bedding plane with variation breadth (modified Pipatpongsa et al. (2022))

2.4 Physical modelling with groundwater seepage

Seepage water poses a significant threat to slopes lying on bedding planes, often leading to collapses near their base, thus greatly impacting slope stability. Seepage-induced landslides are particularly hazardous, especially in mountainous regions, where heavy rainfall, fluctuating water levels, and dam construction contribute to the risk (Budhu & Gobin, 1995; Lu & Godt, 2008). The reduction in soil shear strength due to decreased effective stress caused by seepage is a key factor in slope instability (Brand, 1981). Researchers have explored various aspects of seepage-induced slope failures. Das and Sobhan (1990) derived the factor of safety with respect to strength for infinite slopes, assuming groundwater levels parallel to the bedding plane during seepage. Orense et al.

(2004) examined slope failures induced by water percolation from upslope, emphasizing the significance of soil water content near the base of the slope.

Seepage studies in geotechnical centrifuges have employed various methods. Take et al. (2004) utilized a metering pump to achieve a prescribed flux boundary condition simulating groundwater seepage flow. Deepa and Viswanadham (2009) employed a custom-designed water tank with a perforated wall to simulate seepage effects on soil-nailed slopes. Take et al. (2004) also proposed a slide-to-flow transformation hypothesis as an alternative mechanism, suggesting that the geometry of slip and the characteristics of slowly moving debris could trigger rapid landslides, as depicted in Figure 2-19.

Several interdependent processes occur in slopes affected by seepage:

1. Unsaturated infiltration: Water percolates vertically when rainfall rate on the slope surface is lower than slope permeability.
2. Saturated transmission: Infiltrating water may form a thin seepage layer if upper material permeability exceeds that of the underlying material.

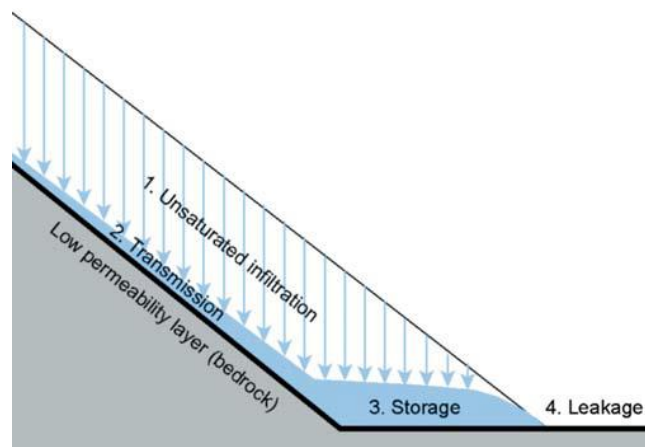


Figure 2-19 Slide-to-flow landslide triggering mechanism(Take et al., 2004)

Take and Beddoe (2014) introduced a finite slope physical model featuring granular soil placed atop impermeable bedrock and exposed to a total groundwater flux (see Figure 2-20). During the experiment, water addition resulted in increased pore water pressure within the granular soil layer. This, in turn, caused a reduction in effective stress until it reached the critical state line (see Figure 2-21), ultimately leading to localized failures at the slope toe due to high pore-water pressure. Figure 2-22 illustrates the slope sliding phenomenon triggered by the rising groundwater seepage.

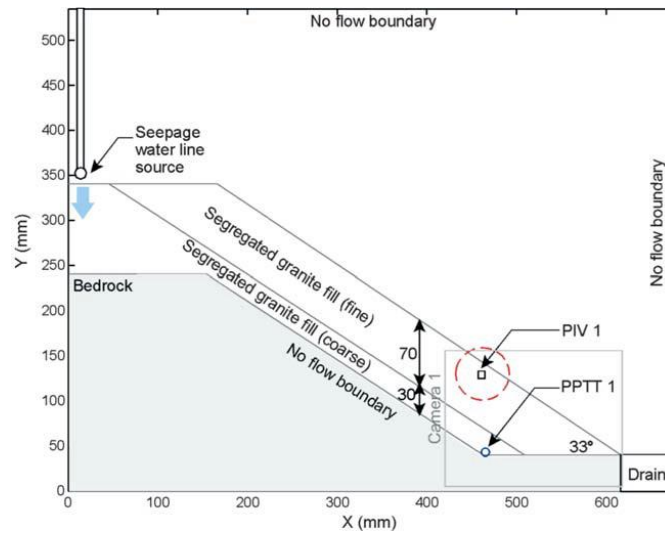


Figure 2-20 Slide-to-flow landslide triggering mechanism model(Take et al., 2004)

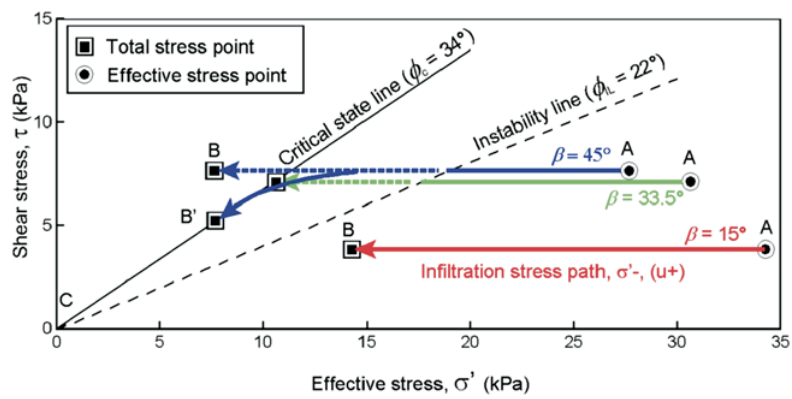


Figure 2-21 Theoretical stress path for infinite slope of varying inclinations.(Take et al., 2004)

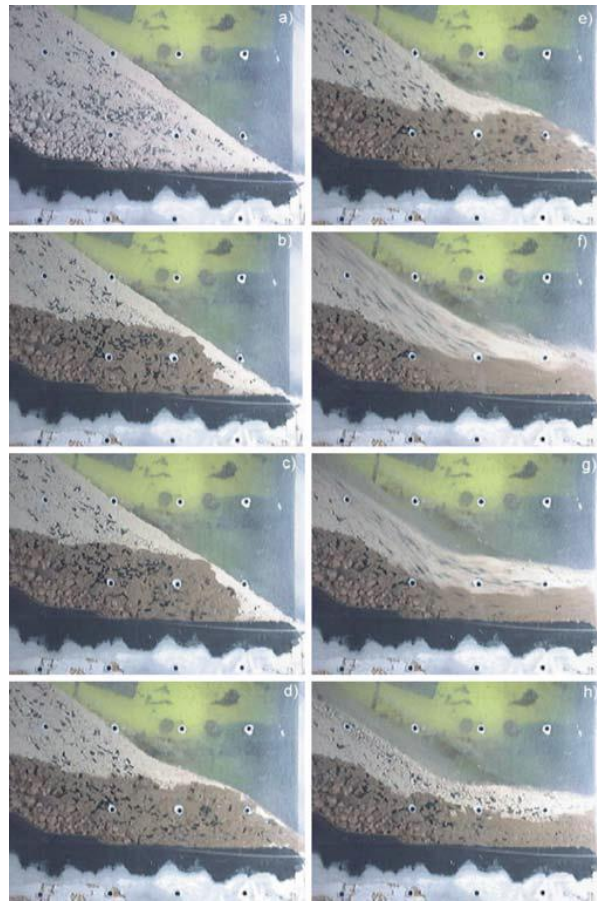


Figure 2-22 Transition from slide to flow in loose fill model (Take et al., 2004)

Additionally, Take and Beddoe (2014) investigated the effects of groundwater seepage on inclined slopes with a base using a centrifugal model, as depicted in Figure 2-23. Groundwater seepage was introduced from the crest of the slope and drained out at the base. Pore-water pressure transducers were installed on the impermeable layer to monitor groundwater seepage during water supplementation. The rate of water supplementation and pore water pressure changes are illustrated in Figure 2-24. As water supplementation increased, the slope model eventually collapsed, leading to immediate changes in pore water pressure. Furthermore, GeoPIV(White et al., 2003) analysis was employed to examine slope deformation based on recorded video footage. The PIV analysis results revealed local failures at the slope toe followed by significant deformation, indicating that groundwater seepage can induce slope instability (see Figure 2-25).

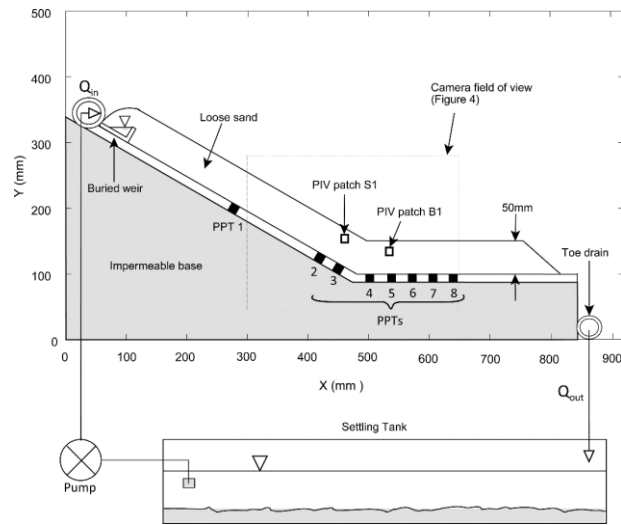


Figure 2-23 Physical model of landslide triggering by seepage-induced (Take & Beddoe, 2014)

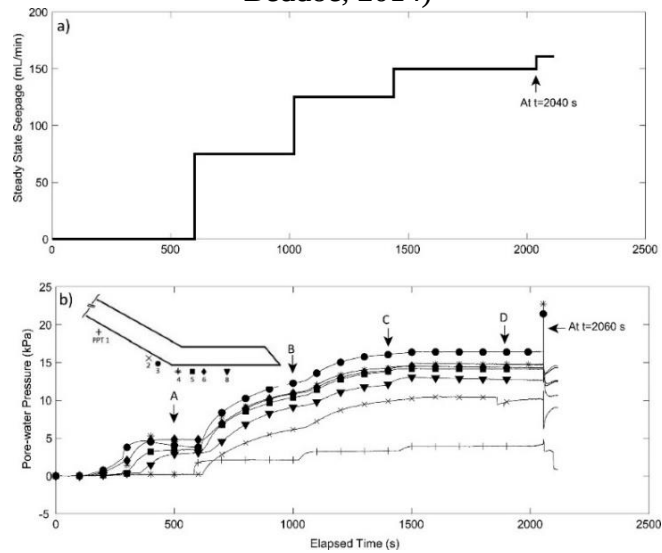


Figure 2-24 (a) Flux boundary condition applied to slope model and (b) resulting pore-water pressure response (Take & Beddoe, 2014)

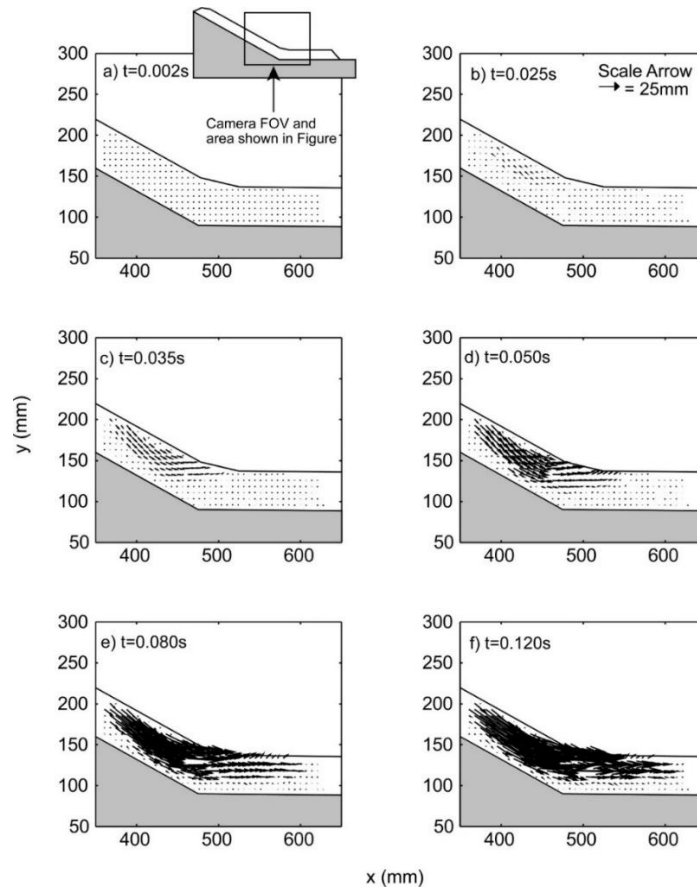


Figure 2-25 Vectors displacement during seepage induced events(Take & Beddoe, 2014)

2.5 Scopes and structure

This chapter initiates by providing an overview of existing research conducted on dip slopes and the diverse triggering mechanisms. Through an in-depth examination of literature, the intention is to draw upon the insights gained from prior studies and integrate them into the framework of this research.

2.5.1 Statement and problem

- In **Section 2.1**, failure mechanisms and stability analysis were reviewed, with emphasis on the orientation of the toe angle. However, further experiments are necessary to validate these failure mechanisms and enhance stability analysis.
- **Section 2.2** discusses the physical modeling of dip slopes with thrust load. The thrust load is used to simulate large boundary conditions, which is crucial due to limitations in laboratory apparatus. However, this method may lead to passive thrust failures occurring behind the thrust load. Therefore, there is a need to study the failure mechanisms and conduct theoretical calculations.

- In **Section 2.3**, dip slopes with lateral confinement are reviewed through site investigation and theoretical calculation. To enhance dip slope stability analysis, the effect of lateral confinement on slope stability is considered, expanding the analysis from two-dimensional (2D) to three-dimensional (3D). Nevertheless, theoretical calculations require experimental validation.
- **Section 2.4** reviews dip slopes with groundwater using a centrifugal model. It explains the impact of groundwater seepage on impermeable bedding planes of dip slopes. However, theoretical calculations are necessary to assess the stability analysis of these slopes under groundwater conditions.

2.5.2 Objective

Based on background in **Chapter 1** and literature review, the following subjects should be studied and investigated to establish a deeper understanding of dip slope failure mechanisms within the field of geotechnical engineering:

- (1) Study the reverse toe effect on failure mechanisms of dip slope by physical modelling
- (2) Development of PIV analysis with statistical approach to evaluate dip slope failure mechanisms
- (3) Investigate the effect of groundwater infiltration on dip slopes by centrifugal modelling
- (4) Explore counterweight mitigation for dip slope stabilization by centrifugal modelling
- (5) Study effect of dynamic loading on dip slope by centrifugal modelling
- (6) Development and validation of slope stability analysis formula

To resolve these subjects, a series of research studies, shown in Figure 2-25, are provided in Chapters 4 to 9. However, **Chapter 3** details the methodology used to achieve the research objectives.

In **Chapter 4**, a total of 12 tests were conducted under 1g conditions to investigate passive thrust near the slope toe in laterally confined slopes on an inclined bedding plane. Additionally, the failure mechanism related to a reverse toe angle was observed in these experiments. PIV analysis with statistical approaches was applied to analyze the failure mode. Additionally, a newly developed minimum coefficient of variation method was utilized to detect slip lines and determine the time of slope failure.

Due to the failure mechanism of dip slopes investigated in Chapter 4, the study will continue to explore the effect of groundwater infiltration on dip slope stability using centrifugal model. In **Chapter 5**, six centrifuge tests were carried out to study slope failure subjected to groundwater infiltration on an impermeable bedding plane. A surcharge load cart acted as a moving boundary, triggering the loading in these

experiments. Moreover, theoretical calculations of slope stability are provided and compared with the experimental results.

In **Chapter 6**, five centrifuge tests focused on counterweight mitigation for dip slope failure due to groundwater infiltration on an impermeable bedding plane. Furthermore, theoretical calculations of slope stability with counterweight fill are provided to analyze stability changes subjected to counterweight fill.

In **Chapter 7**, the behavior of slopes on permeable bedding planes with lateral confinements and toe support was examined through six centrifuge tests, which included two dip slopes without countermeasures and four with countermeasures.

In **Chapter 8**, one centrifuge test investigated the effect of seismic loading on dip slopes. One test applied four seismic waves to determine the impact of seismic loading on slope failure. In this chapter, a combination of pseudo-static and dip slope stability analysis was performed to compare with experimental results.

In **Chapter 9**, the conclusions of this thesis, limitations and recommendations for future work are provided.

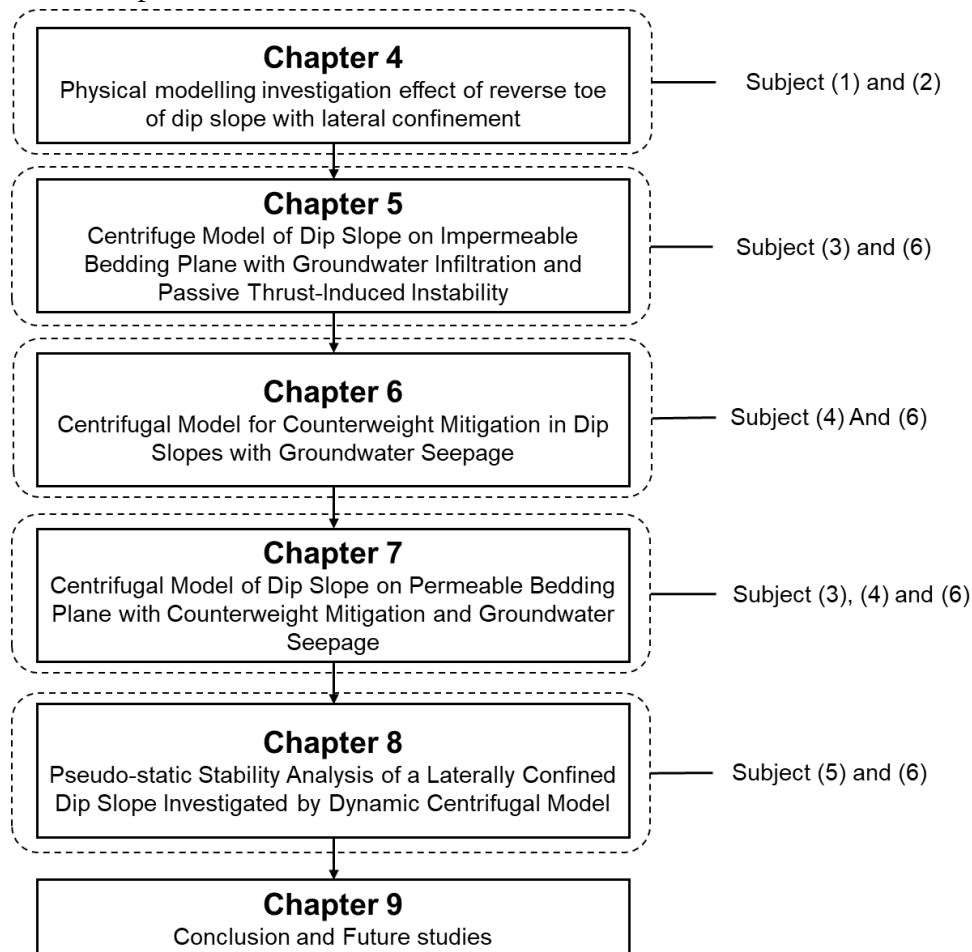


Figure 2-26 Structure of thesis and corresponding objectives

Chapter 3 Methodology

Slope failure mechanisms and deformation behaviours can be examined through various physical modelling methodologies. This research employs both 1g and centrifuge experiments for their distinct advantages. The 1g experiments were selected for their ease of experimental setup control. In contrast, centrifuge modelling was chosen due to its unique ability to replicate field stress levels by creating a hyper-gravity field.

This chapter provides an introduction to the 1g and centrifuge experiments, followed by a detailed description of the model apparatus and instrumentation used in the centrifuge tests. The geotechnical properties of the sand are thoroughly detailed, along with the sand preparation process. Two advanced techniques for measuring earth pressures and sand deformations are then introduced.

3.1 1g physical model

The 1g physical model was employed to investigate passive thrust near the slope toe in laterally confined slopes on an inclined bedding plane. Additionally, the failure mechanism related to a reverse toe angle was observed in these experiments, using humid Silica No. 6 sand as the soil material. Figure 3-1 shows a schematic diagram of the experimental setup. The slope model was compacted with lateral confinements and toe support. The angle of the toe support was varied to investigate the slope's failure mechanism when subjected to different toe angles.

A high-speed camera was used to record video footage during slope collapse. Acrylic plates were used for the bedding plane and the two side lateral supports, allowing the determination of interface properties through a tilting table test. The toe support was made of Teflon sheet. Additionally, a wooden surcharge cart was employed as a trigger by adding dead weight to the cart. Figure 3-2 illustrates the 1g experiment setup in the laboratory, with the high-speed camera positioned next to the lateral support to capture motion from a side view.

The experimental procedure is outlined in Figure 3-3. After constructing the slope model, the loading process began with the surcharge cart. The high-speed camera then captured the slope failure motion. The recorded video was subsequently analysed to determine the vector velocity field using PIV (Particle Image Velocimetry) software.

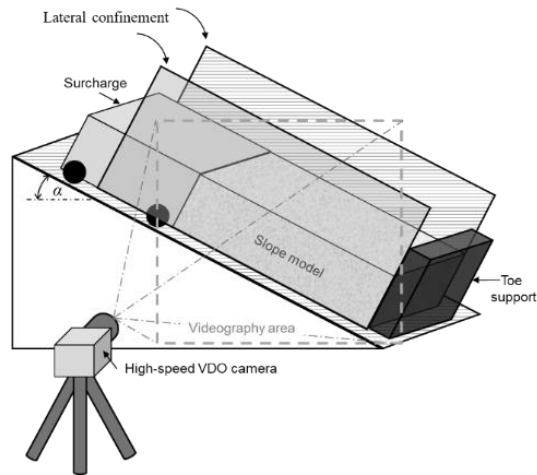


Figure 3-1 Schematic diagram of the experimental setup

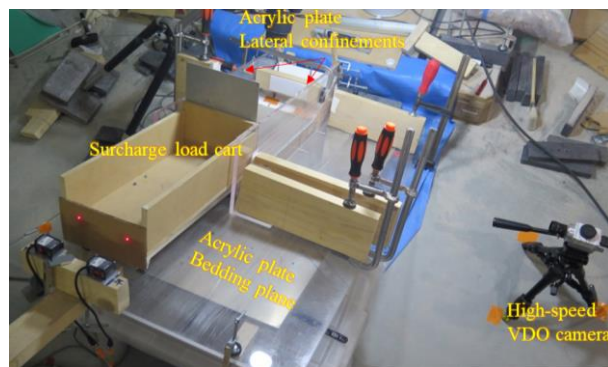


Figure 3-2 1g experimental set up

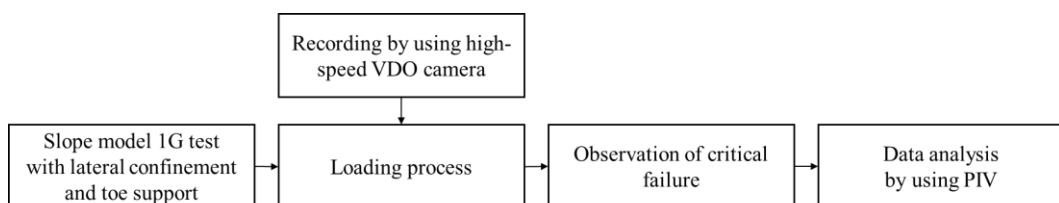


Figure 3-3 Experimental procedure on 1g experiments

3.2 Centrifugal model

3.2.1 Basic theory and scaling laws in centrifuge modelling

Centrifuge modeling has been a widely used technique for testing small-scale models under enhanced gravity fields, effectively replicating field stresses in geotechnical research since the 1960s.

The creation of an enhanced gravity field relies on uniform circular motion, where an object moves around a circle at a constant speed. Although the object's speed remains constant, the change in the direction of its velocity results in acceleration. Figure 3-4 illustrates a black solid circle undergoing uniform motion at a speed of v along a circular path with radius r , accompanied by a set of unit vectors i , j , and k . The position of the solid circle is represented by an angle θ , with the angular speed $\dot{\theta}$ describing the rotation about the center O .

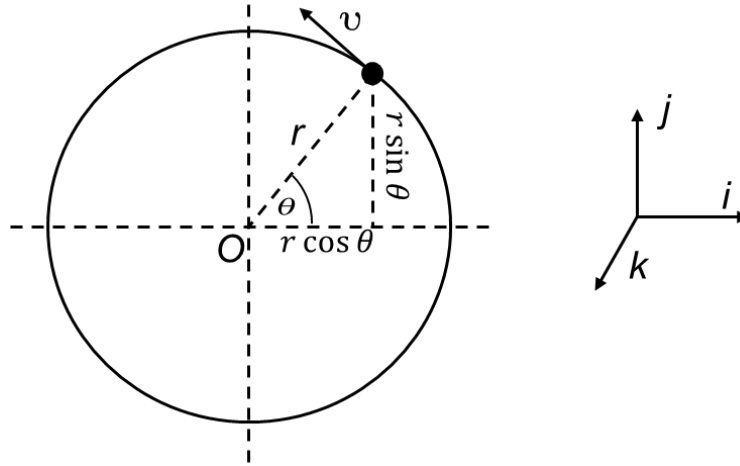


Figure 3-4 Uniform circular motion of solid circle

The position vector of solid circle can be demonstrated as

$$\bar{r} = r \cos \theta \, i + r \sin \theta \, j \quad (3-1)$$

The velocity of the solid circle can be expressed as

$$\bar{v} = \frac{d\bar{r}}{dt} = r(-\sin \theta)\dot{\theta}i + r(\cos \theta)\dot{\theta}j \quad (3-2)$$

The acceleration utilized to solid circle can be expressed by the differentiation of velocity as

$$\bar{a} = \frac{d\bar{v}}{dt} = \left(-\frac{v^2}{r}\right)(\cos \theta i + \sin \theta j) \quad (3-3)$$

Where the minus sign indicates that the acceleration is directed towards the center of the circular path.

Similarly, in geotechnical centrifuge modeling, the container holding the soil body rotates around the central vertical axis of the centrifuge, thereby applying acceleration to the soil. The soil within the container is referred to as the 'model,' while the equivalent system experiencing magnified stresses due to the acceleration is known as the 'prototype'.

The fundamental concept of centrifuge modeling involves testing a 1/N scale model of a prototype within an enhanced gravity field of Ng. Madabhushi (2017) presented a straightforward example to illustrate the principle of centrifuge modeling, as depicted in Figure 3-5. A block structure with mass M and dimensions L×B×H, referred to as the 'prototype,' is placed on a horizontal soil bed and exerts an average vertical stress on the soil, calculated as follows:

$$\sigma_v = \frac{Mg}{L \times B} \quad (3-4)$$

Similarly, the vertical strain mobilized in the soil over a characteristic length α is calculated as:

$$\varepsilon = \frac{\delta\alpha}{\alpha} \quad (3-5)$$

Another block structure, with a mass of M/N³ and dimensions L/N × B/N × B/N × H/N labeled as 'model,' is placed in an enhanced gravity field of Ng. The vertical stress and strain in the soil caused by the block can be calculated as:

$$\sigma_v = \frac{\frac{M}{N^3} \times Ng}{\frac{L}{N} \times \frac{B}{N}} = \frac{Mg}{L \times B} \quad (3-6)$$

$$\varepsilon = \frac{\frac{\delta\alpha}{N}}{\frac{\alpha}{N}} = \frac{\delta\alpha}{\alpha} \quad (3-7)$$

It can be inferred from Eqs 3.4 to 3.7 that the stresses and strains in the model are identical to those at corresponding points of the prototype. This replication of the non-

linear stress-strain relationship of the soil through centrifuge modelling facilitates accurate simulation of geotechnical problems.

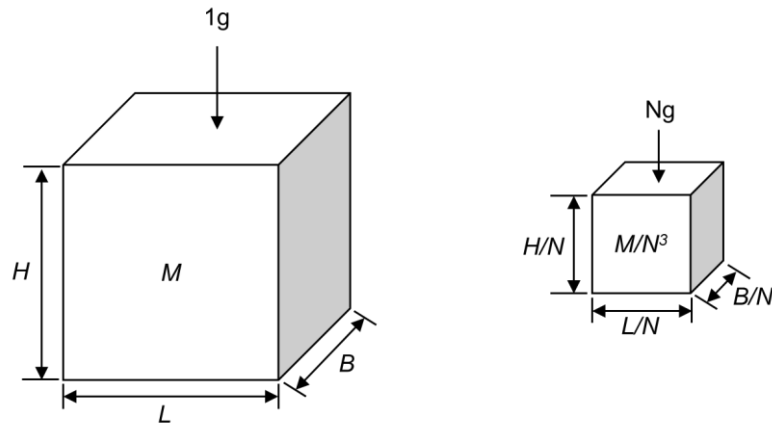


Figure 3-5 Principle of centrifuge modelling (Madabhushi, 2014)

Using a centrifuge model offers the advantage of subjecting a small-scale physical model to accurate stress levels by increasing the G-level, which corresponds to a full-scale field structure (Taylor, 1995). Consequently, the functional relationship between the prototype and the model, known as Scaling laws, can be fulfilled (Schofield, 1980). Table 3-1 illustrates the common scaling laws between the prototype and the model under Ng gravitational acceleration.

Table 3-1 Scaling law of centrifuge modelling

Parameter	Prototype/ Model ($N G$)
Displacement /Length	N
Area	N^2
Force	N^2
Volume	N^3
Velocity	1
Pressure	1
Strain	1
Time (dynamic)	N
Time (consolidation)	N^2

3.2.2 Inherent deficiencies on centrifuge modelling

Figure 3-6 illustrates a variation in centrifugal acceleration along the width direction in the model. As previously discussed, the acceleration magnitude is directly

proportional to the distance from the rotation center, resulting in its constancy along circular paths, as indicated by the blue dashed circles in Figure 3-6. However, points A, B, and C, experiencing the same acceleration, do not lie on a horizontal plane, leading to the enhanced gravity field in the model having a radius of curvature.

To mitigate the impact of this inherent limitation on experimental outcomes, a centrifuge with a sizable radius is utilized, thereby reducing the model's height proportionally.

Figure 3-7 illustrates the third inherent deficiency induced by the Earth's gravity, which acts perpendicular to the centrifugal acceleration directed towards the rotation center. Consequently, the resultant acceleration Rg is inclined at an angle of $\tan^{-1}(1/N)$ to the horizontal.

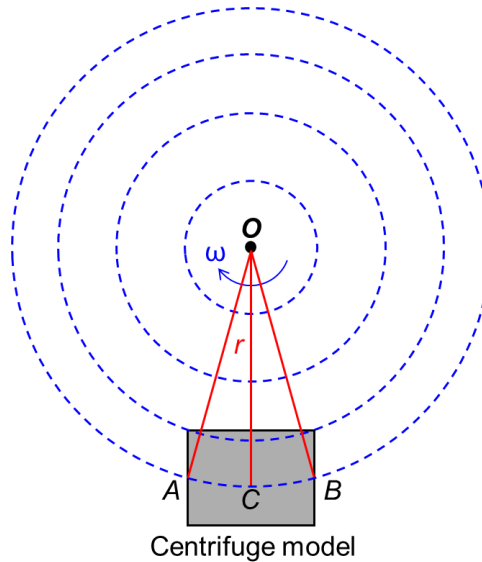


Figure 3-6 Error due to the radial gravity field (Madabhushi G., 2014).

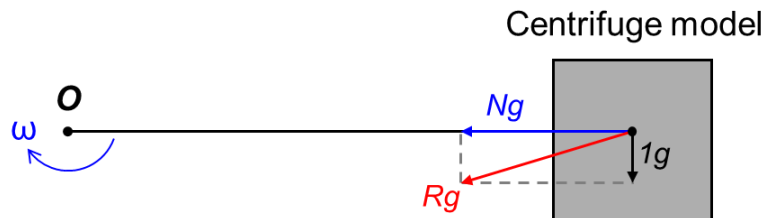


Figure 3-7 Inclination of resultant acceleration

3.2.3 Centrifuge laboratory at Kyoto University

The centrifuge at the Disaster Prevention Research Institute, Kyoto University (DPRI-KU) was employed in this study. Figure 3-8 presents a schematic drawing of the geotechnical centrifuge machine at DPRI. The platform ascends to achieve increasing centrifugal load as the rotational velocity increases. This beam centrifuge has a working radius of 2.5 meters and can achieve a maximum acceleration of 200g. However, in this study, the maximum centrifugal acceleration was limited to 50g due to instrumentation constraints within the centrifuge.

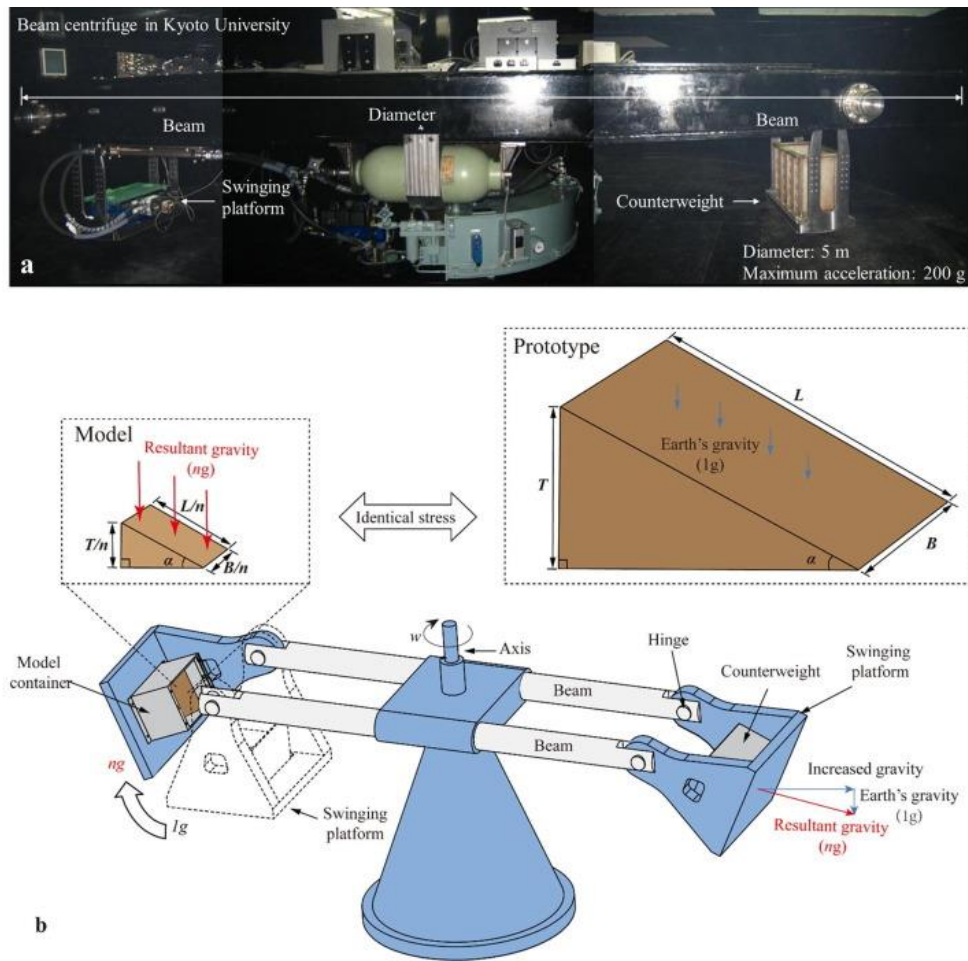


Figure 3-8 Beam centrifuge: (a) beam centrifuge in Kyoto University from a plan view ; (b) schematic view of the beam centrifuge.(Fang et al., 2023)

3.2.4 Geotechnical centrifuge on landslide

The most effective criterion for centrifuge modelling of landslides is the "modelling of models." Initially, the modelling target is determined, guiding the selection of appropriate scaling laws. The type and capacity of the centrifuge machine dictates the g-level, test container design, and landslide model triggering apparatus. Test materials,

slope construction methods, and monitoring instrumentation are then chosen, ensuring accuracy and reliability. Preliminary tests are crucial to identify any abnormal behavior; if issues arise, the design process must be reassessed. Practical limitations during testing should be documented and quantified, and centrifuge landslide tests are conducted under controlled conditions as designed.

Centrifuge modelling is essential for understanding, analyzing, and designing landslides under diverse conditions. Simulations have demonstrated rotational sliding failure in soil slopes and toppling failure in rock slopes by increasing centrifugal acceleration. Rainfall simulations using spray nozzles have shown how changes in pore water pressure can cause soil slope failures. Shaking tables have been used to study earthquake-induced landslides. Models simulating landslides due to water level changes and permafrost thawing have been created using water pumps and warm air systems, revealing failure modes such as progressive failure, mudflow, flow slides, solifluction, and detachment slides. Landslide excavation has been simulated using a soil bag, pulley system, and line-in-flight cut, with additional conditions created using a releasing gate, tilting table, and heater.

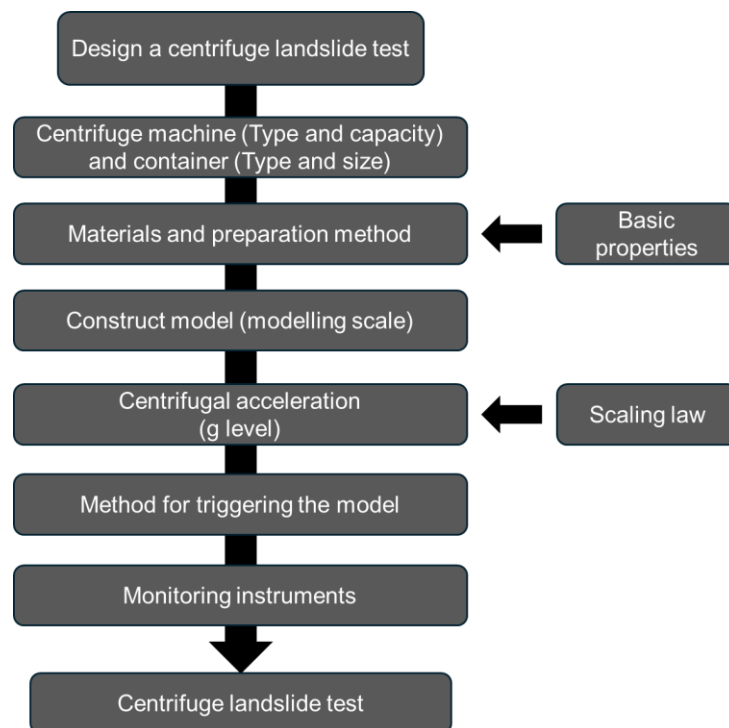


Figure 3-9 Schematic for designing the centrifuge landslide test.(modified from Fang et al. (2023))

3.3 Model apparatus

Geotechnical centrifuges typically utilize a soil chamber to contain the soil during the application of centrifugal acceleration. In this experiment, however, a dip slope was constructed on a slope frame designed to accommodate specific slope geometries. Figure 3-10 illustrates a slope lying on a bedding plane with a surcharge cart serving as a moving boundary. Groundwater seepage was supplied by a top supply tank, with water added to the model via pipes within the soil chamber. This water would drain at the toe of the slope into a bottom storage tank, from which it could be recycled to the top supply tank using a pump and a water level instrument.

Figure 3-11 shows the centrifuge model container, which includes the top supply tank, slope frame, and surcharge cart. Water supplementation was controlled using solenoid valves, while the surcharge cart was controlled by a motor designed to accommodate the surcharge weight. Figure 3-12 depicts the top supply tank, water supply system, and motor. The slope frame included a bedding plane made of porous stone and a toe support made of an aluminum plate, as shown in Figure 3-13. The bedding plane could be covered with a Teflon sheet to serve as a permeable material. Water is supplied to the bedding plane through a pipe (see Figure 3-14) connected to a top supply tank, with flow controlled by a solenoid valve. Additionally, two pipes are provided to increase the water supply quantity. The bottom of the slope frame housed the bottom storage tank, which contained drainage water from the model and the water drainage and recycling system, as illustrated in Figure 3-15.

To control the surcharge cart, a motor connected with a cable was used. A load cell was employed to read the drag force during the increase of centrifugal acceleration and its application to the slope, as shown in Figure 3-16. The water supplementation solenoids and the surcharge motor were connected to control boxes, which were linked to a computer program for precise control, as shown in Figure 3-17.

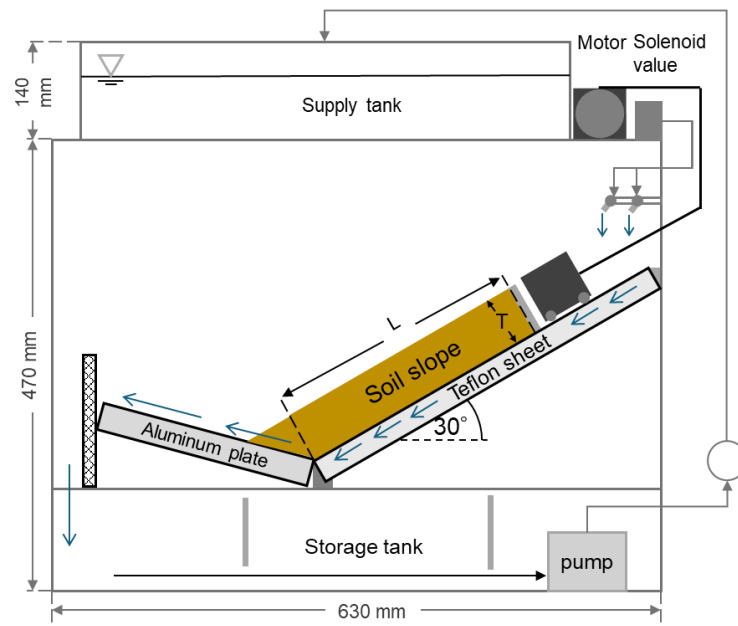


Figure 3-10 Schematic diagram of triggering system of centrifuge model



Figure 3-11 Front of centrifuge model container

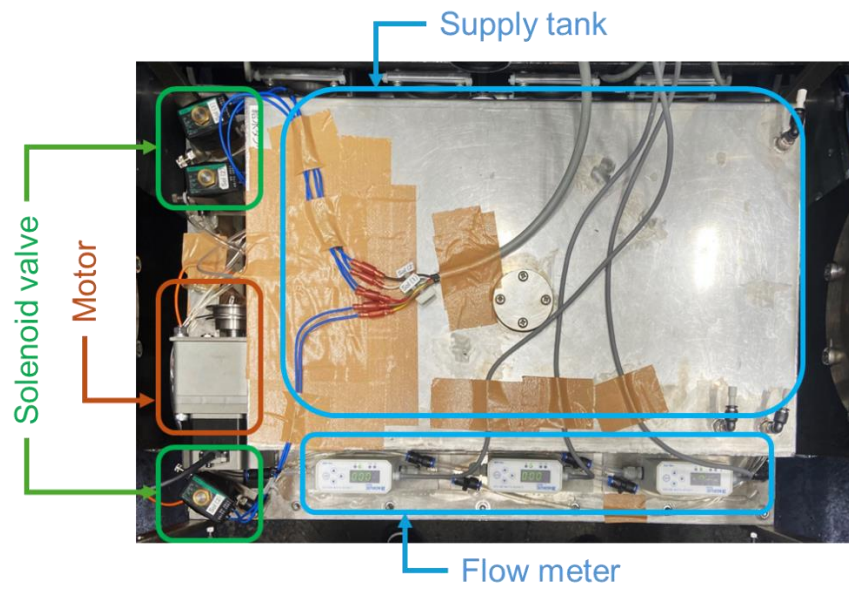


Figure 3-12 Top supply tank cover centrifuge model container

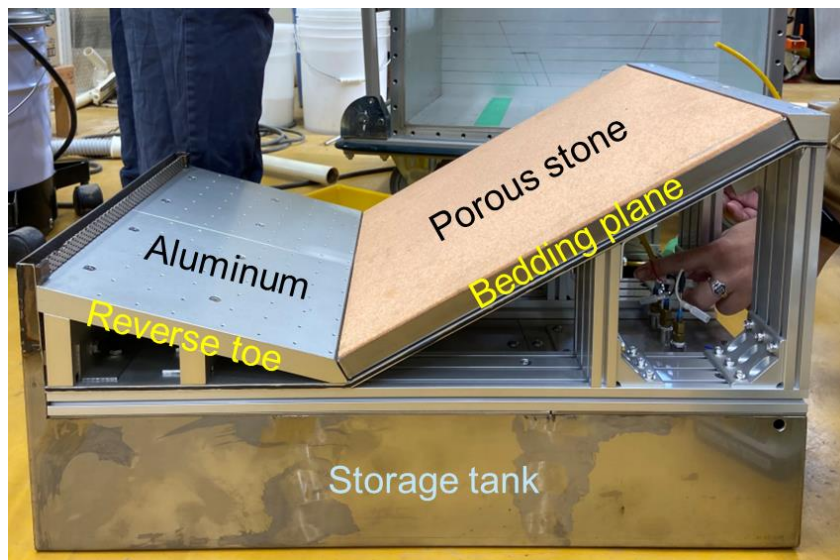


Figure 3-13 Slope frame of centrifuge model and bottom storage tank

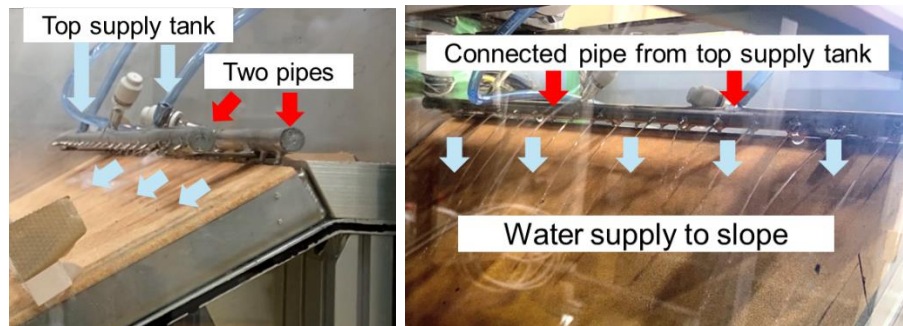


Figure 3-14 Water supplementation

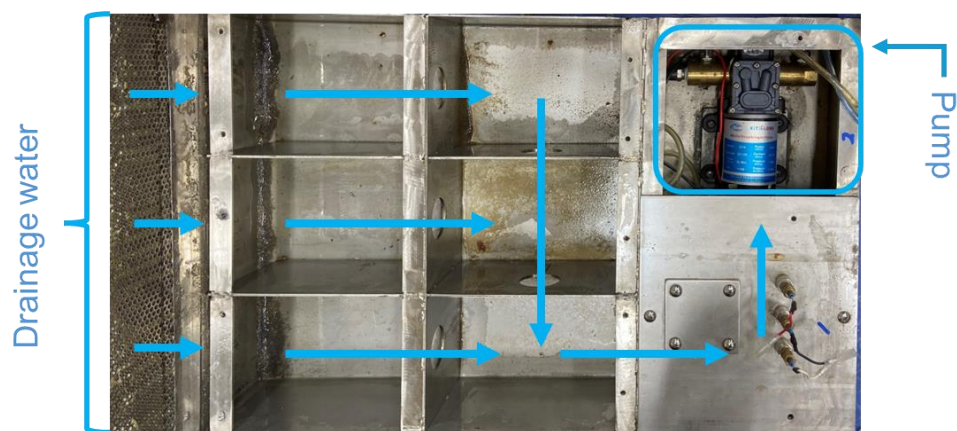


Figure 3-15 Drainage system of bottom storage tank and pump

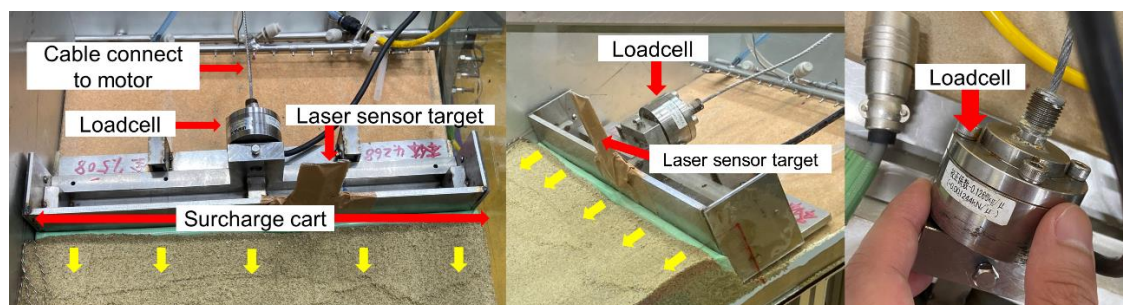


Figure 3-16 Surcharge cart and loadcell

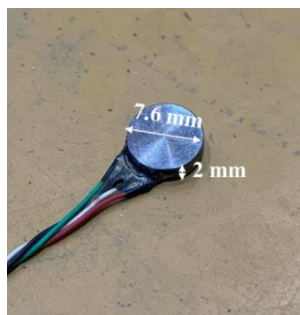


Figure 3-17 Control boxes of water supplementation and surcharge controller.

3.4 Measuring instruments

3.4.1 Pressure gauge

To record and monitor earth pressures in the slope model, a set of miniature pressure gauges was installed at designated locations inside the slope during the preparation process. These pressure gauges, produced by Tokyo Sokki Research Factory, are of type PDA-500KPB, with a capacity of 500 kPa. The dimensions of the pressure gauges are 7.6 mm in diameter and 2 mm in thickness, as shown in Figure 3-18.



a) Dimension



b) Picture

Figure 3-18 Pressure gauge

3.4.2 Laser sensor

Laser sensors were used to measure displacement at various positions to achieve different objectives, such as investigating upheaval at the basal support and monitoring slope displacement. Two types of laser sensors were utilized in this study, as shown in Figure 3-19: IL-065 and IL-100, with effective ranges of 65 mm and 100 mm, respectively. These sensors are produced by KEYENCE Corporation.



a) IL-065



b) IL-100

Figure 3-19 Laser sensor

3.4.3 Pore-water pressure transducer

The slope failure test along the inclined plane will be induced using water. Therefore, this test requires measuring pore water pressure within the slope using a pore water pressure gauge. The gauge is installed on the inclined plane at predetermined positions during the test preparation process. The pressure gauge is manufactured by SSK, model P306A-2, as shown in Figure 3-20.



a) Pore water pressure transducer

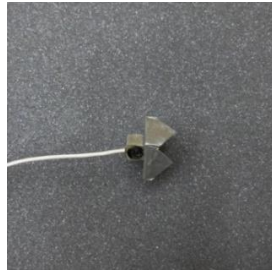


b) Cable of pore water pressure transducer

Figure 3-20 Pore water pressure transducer

3.4.4 Accelerometer

The slope failure test along the inclined plane will result in sliding failure. Accelerometers will be installed within the slope to measure displacement in some tests as a replacement for laser sensors, due to the specific installation requirements of the laser sensors. These accelerometers are placed on the inclined plane at predetermined positions during the test preparation process. The accelerometers used are manufactured by SSK, model A6H-50, as depicted in Figure 3-21.



a) Accelerometer head



b) Cable of Accelerometer

Figure 3-21 Accelerometer

3.5 Experimental material

3.5.1 1g test

Silica sand No.6, with the basic material properties listed in Table 3-2, was used to conduct the reproduced test based on Techawongsakorn (2014). The material properties in this experiment have slight variations from the previous study. Silica sand No.6 was chosen to facilitate the application of PIV (Particle Image Velocimetry) analysis, as recommended by Khosravi et al. (2011) and Ohta et al. (2010).

Table 3-2 Basic properties of Silica sand No. used in 1g test (Fang, 2019)

Parameter	Silica sand No. 6
Water content (w)	10.0 %
Average particle diameter (D_{50})	0.32 mm
Uniformity coefficient (U_c)	1.36
Specific gravity (G_s)	2.643
Bulk unit weight (γ)	13.68 kN/m ³
Maximum void ratio (e_{max})	1.130
Minimum void ratio (e_{min})	0.699
Relative density (D_r)	10.6 %
Unconfined compressive strength (σ_c)	2.1 kPa
Friction angle (ϕ)	33.2°
Apparent cohesion (c)	0.57 kPa

Interface properties were obtained from a tilting table test, following Fang's (2019) experiment. Table 3-3 summarizes the interface shear strength parameters between silica sand No.6 and acrylic plate, as well as between silica sand No.6 and Teflon sheet.

Table 3-3 Summary of interface strength parameters

Parameter	Average value
Friction angle of the roller cart (ϕ_r)	6.1°
Interface friction angle of humid sand on acrylic plate (ϕ_i)	29.4°
Interface adhesion of humid sand on acrylic plate (c_i)	0.26 kPa
Interface friction angle of humid sand on Teflon plate (ϕ_t)	21.0°
Interface adhesion of humid sand on acrylic plate (c_t)	0.03 kPa

3.5.2 Centrifuge test

Using poorly graded soil (Silica sand No. 6) resulted in water draining out during the increase in centrifugal acceleration, thereby altering the modeling material properties. In contrast, well-graded soils like Hiroshima sand and Edosaki sand led to a clear slope failure while maintaining their original water content. Therefore, well-graded soils (Hiroshima sand and Edosaki sand) are recommended for centrifugal modeling. Additionally, before using these materials in experiments, they were sieved to remove particles larger than 2 mm in diameter to prevent the influence of large particles on the behavior of the centrifuge-scale model (Weiler and Kulhawy, 1982). In this study, two different experiments were conducted to investigate both static and dynamic behaviors.

Static experiment: The fundamental properties of Hiroshima sand used in a static centrifugal model are presented in Table 3-3. The centrifuge model was divided into two parts: one with a slope resting on an impermeable bedding plane and the other on a permeable bedding plane. A Teflon sheet was installed as the impermeable bedding plane material to ensure low interface friction. A summary of the interface shear strength between Hiroshima sand and the Teflon sheet is provided in Table 3-5. Additionally, a porous stone, with a coefficient of permeability (k) of 5×10^{-5} m/s, surpasses the permeability of Hiroshima sand. Thus, it can be concluded that water flows more easily through the porous stone than through the soil model. The interface shear strength parameters between Hiroshima sand and the porous stone are detailed in Table 3-6.

Table 3-4 Basic properties of Hiroshima sand

Dry maximum density (ρ_{dmax})	1.747 g/cm ³
Water content (w)	8.0%
Degree of compaction (D_c)	90%
Coefficient of permeability (k)	2.4×10^{-5} m/s
Friction angle (ϕ)	33.13°
Apparent cohesion (c)	0.309 kPa

Table 3-5 Summary of interface shear strength for Hiroshima sand

Friction angle of the roller cart on Teflon sheet (ϕ_r)	10°
Interface friction angle of humid sand on Aluminum(ϕ_i)	25.4°
Interface adhesion of humid sand on Aluminum (c_i)	0.22 kPa
Interface friction angle of humid sand on Teflon sheet (ϕ_t)	24.23°
Interface adhesion of humid sand on Teflon sheet (c_t)	0.006 kPa

Table 3-6 Interface shear strength between Hiroshima sand and Porous stone

Friction angle of the roller cart on Porous stone (ϕ_r)	10°
Interface friction angle of humid sand on Porous stone (ϕ_t)	37.98°
Interface adhesion of humid sand on Porous stone (c_t)	0.177 kPa

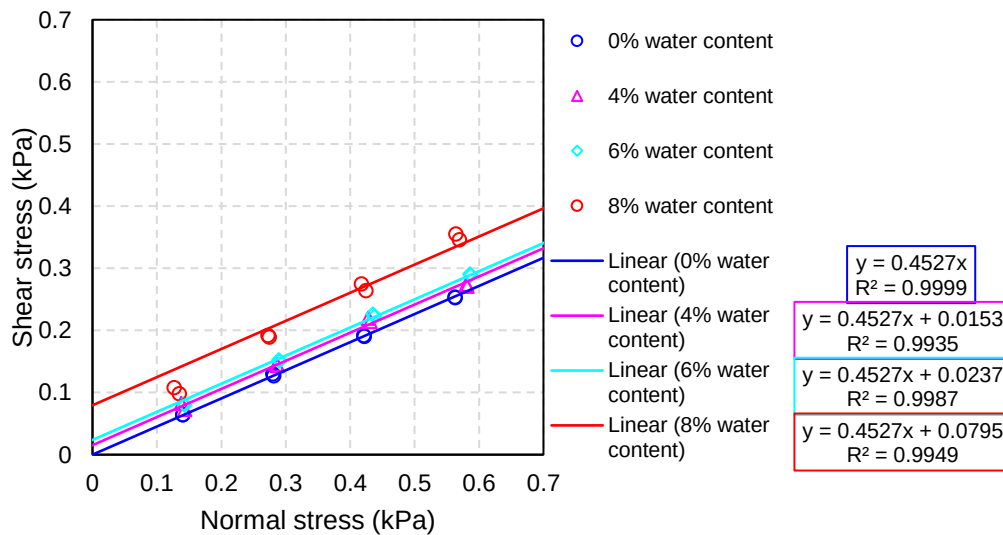


Figure 3-22 Normal stress and shear stress of interface friction between Hiroshima sand and Teflon sheet

Dynamic experiment: The basic properties of Edosaki sand used in an slope model in dynamic case are shown in Table 3-7. The slope was covered by a Teflon sheet for the purpose that the low friction interface plane can be demonstrated. Interface friction angle and apparent adhesion are the parameters between Edosaki sand and a Teflon sheet. Moreover, a slip test was conducted to measure the interface friction angle and apparent adhesion between sand and a Teflon sheet.

Table 3-7 Basic properties of Edosaki sand used in a slope model

Optimum water content (w_{opt})	10 %
Maximum dry density ($\rho_{d\ max}$)	1700 kg/m ³
Unconfined compressive strength (σ_c)	5.75 kN/m ²
Interface friction angle (ϕ_i)	14.7 °
Apparent adhesion (c_i)	0.2 kN/m ²
Degree of compaction (D_c)	90 %
Internal friction angle (ϕ)	40.9°

3.6 Model preparation

3.6.1 1g test

The 1g experiment utilized Silica sand No. 6, whose material properties are presented in Table 3-2. For the preparation of the model, the soil was adjusted to a 10% water content. Subsequently, two lateral supports, a toe support, and a wooden plate were arranged as molds to confine the geometrical boundaries. The adjusted soil was then poured into the prepared mold and compacted to achieve a compaction degree of 90%. To regulate the degree of compaction, a drawing line was employed. Figure 3-23 illustrates the physical setup of the 1g experiment after the completion of the preparation process, depicting the slope angle α , toe angle β , thickness T , length L , and width B . The lateral supports consisted of acrylic plates, which were transparent to facilitate clear observation of the failure process during slope collapse and enabled the recording of video footage.

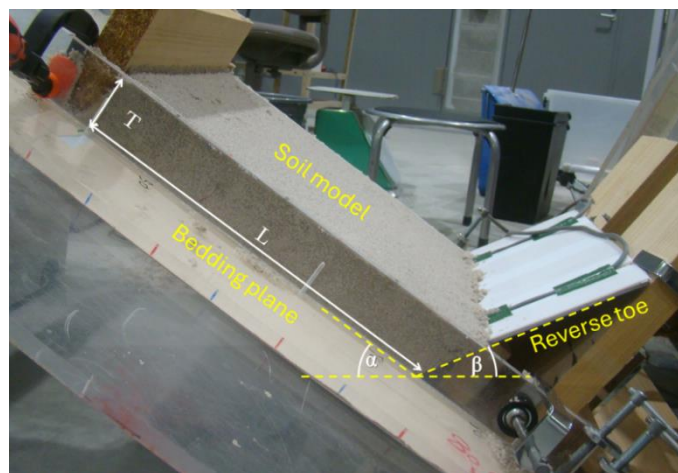


Figure 3-23 1g experimental preparation

3.6.2 Centrifuge test

For both static and dynamic centrifuge model preparations, the same method was employed. The process began with adjusting the sand to achieve a water content of 8%. The soil properties of Hiroshima and Edosaki sand are detailed in Table 3-4 and Table 3-7, respectively. Subsequently, instrumentation was meticulously installed on Teflon sheets or porous stones and aluminum plates, as illustrated in Figure 3-24(a). Following this, the soil chamber underwent rotation by a crane to orient the bedding plane horizontally, facilitating compaction.

Once the bedding plane was properly aligned, a wooden plate was affixed to constrain the model to the desired slope length. The adjusted sand was then carefully placed into the prepared mold, as depicted in Figure 3-24, and compacted within delineated lines to regulate the compaction degree, as shown in Figure 3-24(b). Upon completion of compaction, the soil chamber was rotated back to its original position, and laser sensors were installed. Figure 3-24(c) shows the model after preparation for the static experiment, while Figure 3-25 depicts the model after preparation for the dynamic experiment.

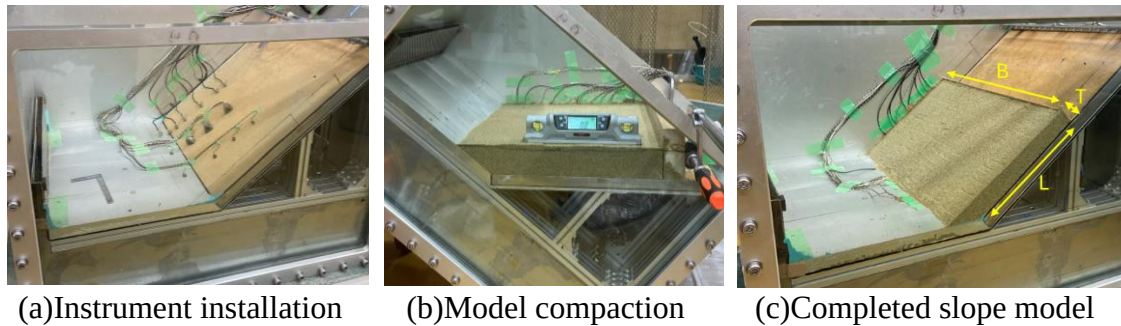


Figure 3-24 Preparation of centrifugal model



Figure 3-25 Centrifugal model for dynamic experiment

3.7 Particle image velocimetry

During the loading process of the experiments, the slope collapsed within a mere second, prompting the installation of a high-speed camera to record video footage of the event. This high-speed camera boasts impressive capabilities, capturing video at a rate of 200 frames per second and with a resolution of 800x600 pixels. Images of the slope profile were captured through the transparent side wall of the model chamber. The camera's field of view was specifically targeted to capture the behavior of the slope model, ensuring that critical details of the failure process were recorded with precision. As a result, the movement of the slope was captured with remarkable clarity and fluidity, providing valuable insights into the dynamics of slope failure.

However, despite the high-quality footage obtained from the recorded video and captured images, they were insufficient in clearly depicting the intricate movement of the slope during the failure process, as shown in Figure 3-26. To address this limitation and gain a deeper understanding of the slope's behavior, Particle Image Velocimetry (PIV) analysis will be employed in this research. PIV analysis allows for the visualization and quantification of slope movement through displacement and velocity vectors. By analyzing the displacement and velocity of landslide events using the particle image velocimetry (PIV) image analysis code PIVlab (Thielicke & Sonntag, 2021), researchers can obtain a more detailed and comprehensive understanding of the dynamics involved in the failure process.

The PIV image analysis technique tracks subsets of the image matrix across a series of recorded frames. To ensure unique identification of these subsets between frames, it is crucial for them to contain sufficient image texture. PIV algorithms compare groups of pixels bound by an interrogation window (IW).

Keane and Adrian (1992) recommend employing an average of 10 or more particles per interrogation window to maximize the accuracy of PIV algorithms. Additionally, they recommended that particle displacement should not exceed 25% of the IW length. Assuming a minimum IW length of 16 pixels, this results in an IW area of 256 pixels, or approximately 1000 particles, which satisfies Keane and Adrian's suggested guideline, as illustrated in Figure 3-27.

Additionally, a region of interest (ROI) and an objective mask were utilized in the Particle Image Velocimetry (PIV) analysis to concentrate on the specific slope segment. Figure 3-28 presents the ROI and objective mask of test A2, where active vectors denote PIV analysis values, while inactive vectors denote areas not subjected to PIV analysis. Following the verification and validation of the PIV analysis, adjustments to the geometry of the objective mask will be made to facilitate its application to all experimental tests.

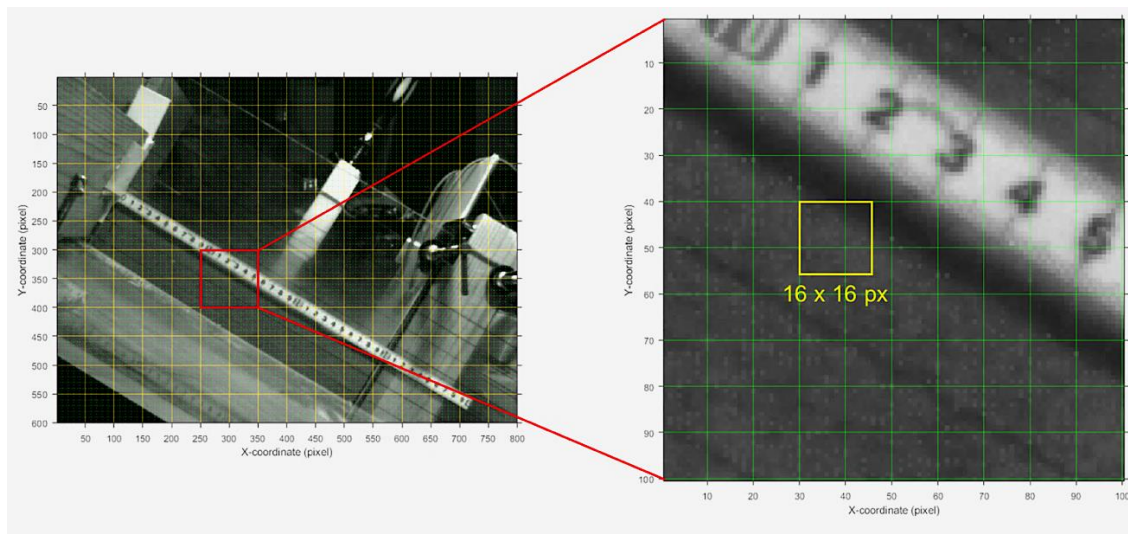
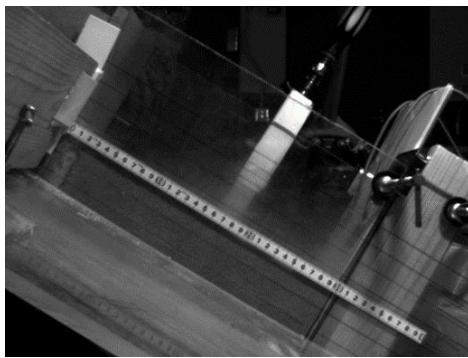
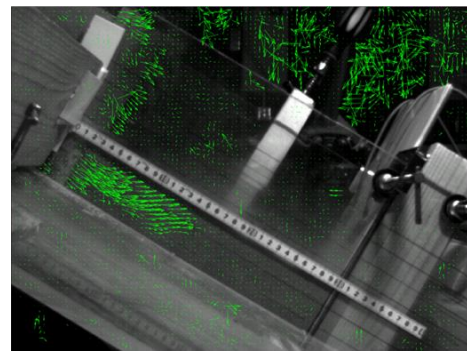


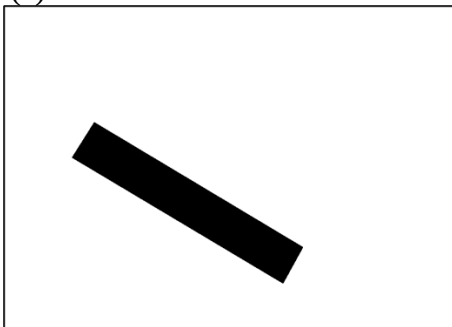
Figure 3-26 Interrogation window of PIV analysis



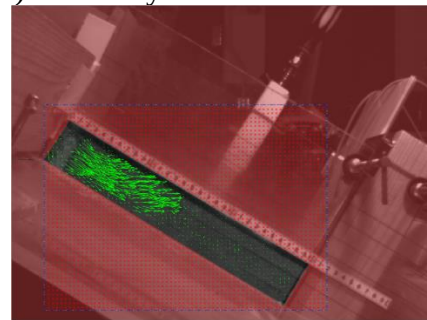
(a) Record video from HS-camera



(b) PIV analysis result without mask



(c) Mask capture



(d) Final result from PIV analysis

Figure 3-27 Application of mask, ROI and PIV analysis

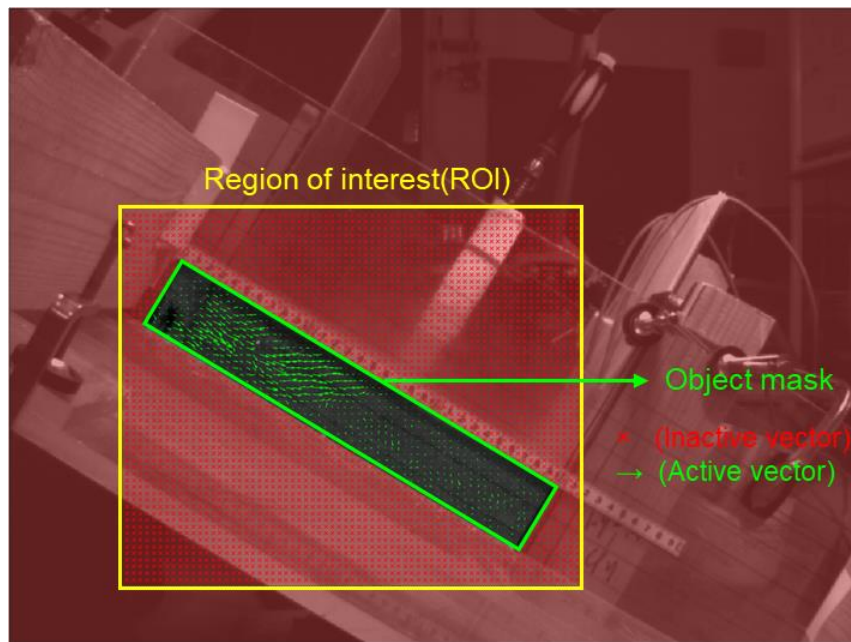


Figure 3-28 Diagram of PIV calculation

Chapter 4 Study the reverse toe angle effect of dip slope failure mechanism

This study delves into the failure mechanisms of slopes situated on bedding planes with lateral confinements and toe support. Leveraging insights from previous research endeavors (Take et al. (2004); Take and Beddoe (2014)) which have extensively employed physical models to investigate a wide spectrum of slope failures and landslides, this paper employs 1g physical modelling to explore the reverse toe effect on slope failure modes. Thrust load is employed to induce slope failure, conceptualized as a large boundary condition. Rankine's passive wall movement theory, commonly utilized to evaluate failure zones, is defined by the passive wedge angle $\theta=45-\phi/2$ (Rankine, 1857). However, it is necessary to study and compare the passive failure zone of a dip slope under thrust load with Rankine's theory. Consequently, this chapter presents a novel Particle Image Velocimetry (PIV) analysis, utilizing the minimum coefficient of variation method, to detect slip lines that demarcate the failure zone.

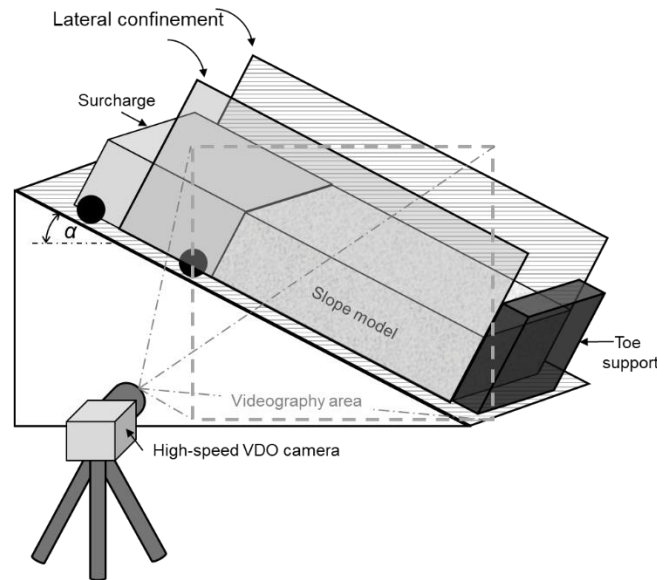


Figure 4-1 schematic of slope lying on bedding plane with lateral and toe support

The experimental setup entails constraining the slope model with two side supports and a toe support, as illustrated in Figure 4-1. To accomplish this goal, a series of 12 physical model tests were conducted, varying the reverse toe angles. Silica sand no.6 was used to construct the physical slope models, ensuring consistency in material

properties. The interface shear strength parameters, such as the friction angle of the roller cart, were determined and summarized in Table 1, based on multiple sliding tests. Additionally, the material employed for lateral support mirrored that used for the bedding plane, ensuring uniformity in interface properties.

The initiation of all physical models was facilitated by a surcharge cart, acting as a movable boundary. Despite sharing common dimensions—a thickness (T) of 60mm, a length (L) of 300mm, and a breadth (B) of 209mm—the tests were segregated into two series. One series featured slopes inclining at 30 degrees, while the other comprised slopes inclining at 50 degrees. Each series explored various toe angles, delineated in Table 3. Specifically, the six tests with a slope inclination of 30 degrees encompassed a range of toe angles from 60 to 0, reduced by 20 degrees. Conversely, the six tests with a slope inclination of 50 degrees spanned a range of toe angles from 40 to -20, also reduced by 20 degrees.

Table 4-1 Maximum surcharge and pressure in laterally confined slope models subject to a toe support

Test №	Thickness T (mm)	Length L(mm)	Breadth B (mm)	Slope angle α (°)	Toe angle β (°)
A1 A2 A3	60	300	209	30	60
B					40
C					20
D					0
E1 E2				50	40
F					20
G1 G2					0
H					0
					-20

4.1 Observation of failure

During the surcharge loading process, precise monitoring of slope movements was achieved through the utilization of a high-speed camera, boasting a remarkable recording capability of 200 frames per second. Positioned strategically, as depicted in Figure 4-2, the high-speed camera captured the entirety of the experimental setup, ensuring comprehensive documentation of the slope behavior.

Of particular interest was the failure process of the slope with toe support, encapsulated in the sequential images from Figure 4-2(a) to (d). Commencing from the

initial application of the surcharge at $t = 0.00\text{s}$, the progression unfolds, culminating in the final step at time $t = 1.89\text{s}$. These images provide a vivid depiction of the dynamic evolution of the slope failure mechanism.

Intriguingly, the observed deformations exhibited distinct characteristics. Figure 4-2(a) and (b) showcase subtle yet discernible shifts, indicative of a sliding mode during the initial stages of slope yield. Contrastingly, the pronounced distortions captured in Figure 4-2(c) and (d) portray a more dramatic upheaval, signaling the collapse of the slope precipitated by the toe breakout.

Thus, the comprehensive visual documentation afforded by the high-speed camera offers invaluable insights into the intricate dynamics of slope failure mechanisms, shedding light on the underlying processes governing slope behavior under surcharge loading conditions.

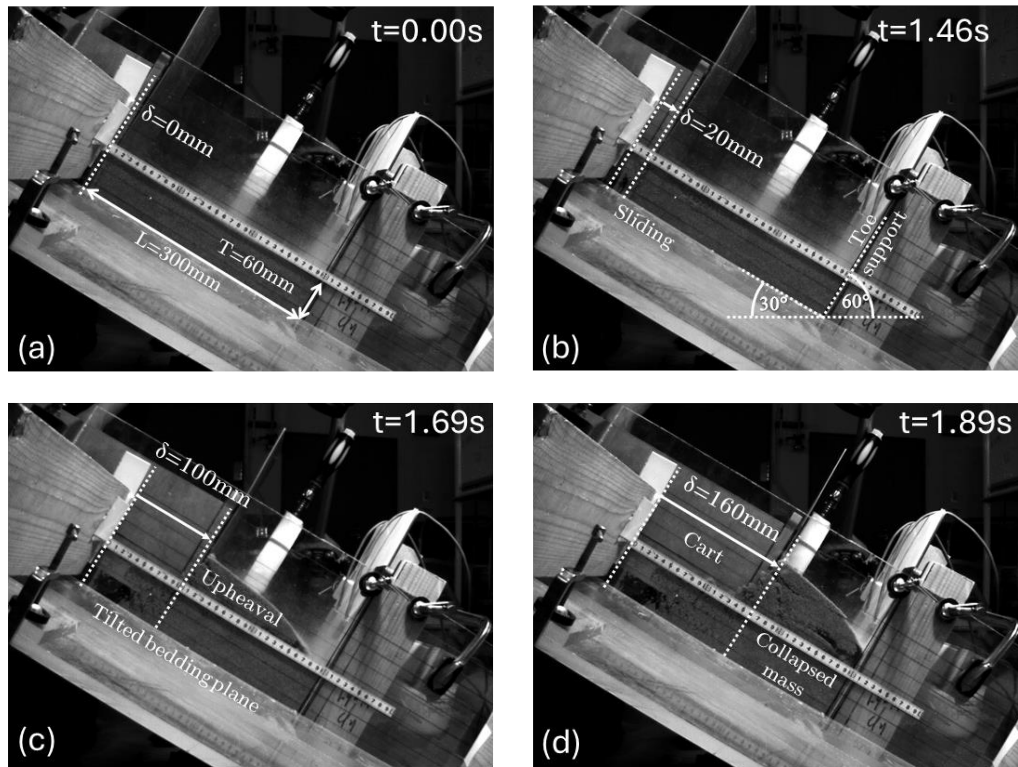


Figure 4-2 Failure observation during experiment

According to Pipatpongsa et al. (2023), in the study on the effect of reverse toe influence on the failure mode of slopes lying on a bedding plane, the results indicate that the reverse toe angle significantly influences the mode of failure. Specifically, when the summation of the slope angle (α), reverse toe angle (β), and friction angle of the toe support (ϕ_i) exceeds 90 degrees, it leads to thrust failure. This failure mode is characterized by significant movement from the crest towards the middle of the slope.

Conversely, if the summation is less than 90 degrees, the failure mode tends to be toe sliding.

4.2 Particle image velocimetry

During the loading process of the experiments, the slope collapsed within a mere second, prompting the installation of a high-speed camera to record video footage of the event. This high-speed camera boasts impressive capabilities, capturing video at a rate of 200 frames per second and with a resolution of 800x600 pixels. Images of the slope profile were captured through the transparent side wall of the model chamber. The camera's field of view was specifically targeted to capture the behavior of the slope model, ensuring that critical details of the failure process were recorded with precision. As a result, the movement of the slope was captured with remarkable clarity and fluidity, providing valuable insights into the dynamics of slope failure.

However, despite the high-quality footage obtained from the recorded video and captured images, they were insufficient in clearly depicting the intricate movement of the slope during the failure process, as shown in Figure 4-2. To address this limitation and gain a deeper understanding of the slope's behavior, Particle Image Velocimetry (PIV) analysis will be employed in this research. PIV analysis allows for the visualization and quantification of slope movement through displacement and velocity vectors. By analyzing the displacement and velocity of landslide events using the particle image velocimetry (PIV) image analysis code PIVlab, researchers can obtain a more detailed and comprehensive understanding of the dynamics involved in the failure process, Senatore et al. (2013) ;Thielicke and Sonntag (2021)

The PIV image analysis technique tracks subsets of the image matrix across a series of recorded frames. To ensure unique identification of these subsets between frames, it is crucial for them to contain sufficient image texture. PIV algorithms compare groups of pixels bound by an interrogation window (IW).

Keane and Adrian (1992) recommend employing an average of 10 or more particles per interrogation window to maximize the accuracy of PIV algorithms. Additionally, they recommended that particle displacement should not exceed 25% of the IW length. Assuming a minimum IW length of 16 pixels, this results in an IW area of 256 pixels, or approximately 1000 particles, which satisfies Keane and Adrian's suggested guideline, as illustrated in Figure 4-3.

The experiments analyzed soil motion under conditions where the soil slope was displaced. The maximum speed of PIV results was 25 mm/s, which aligns with the criterion that displacement should not exceed 25% of the IW length (448 mm/s). The results of the scatter plot between horizontal and vertical velocity vectors are depicted in

Figure 4-5. Therefore, PIV analysis can be effectively utilized in the recorded video of the experiment.

Additionally, a region of interest (ROI) and an objective mask were utilized in the Particle Image Velocimetry (PIV) analysis to concentrate on the specific slope segment. Figure 4-4 presents the ROI and objective mask of test A2, where active vectors denote PIV analysis values, while inactive vectors denote areas not subjected to PIV analysis. Following the verification and validation of the PIV analysis, adjustments to the geometry of the objective mask will be made to facilitate its application to all experimental tests.

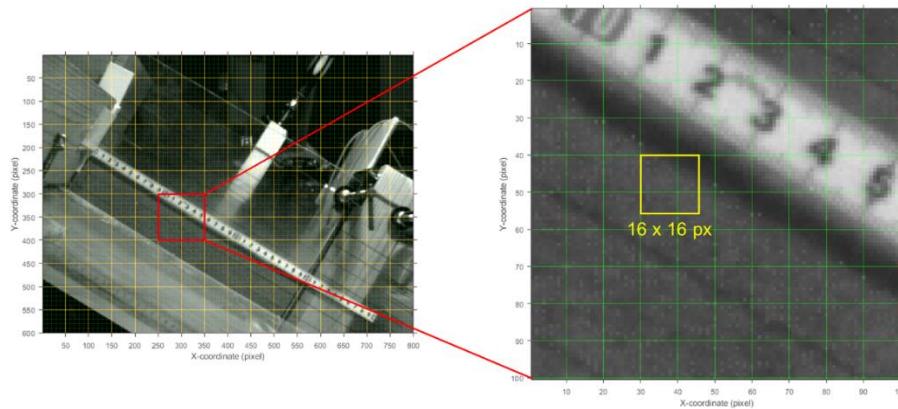


Figure 4-3 Pixels on recorded video of dip slope

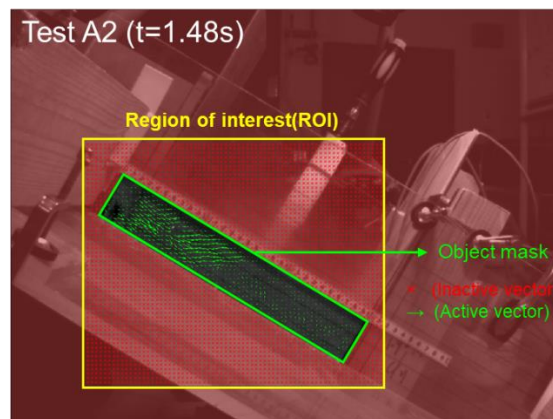


Figure 4-4 Application of mask and ROI

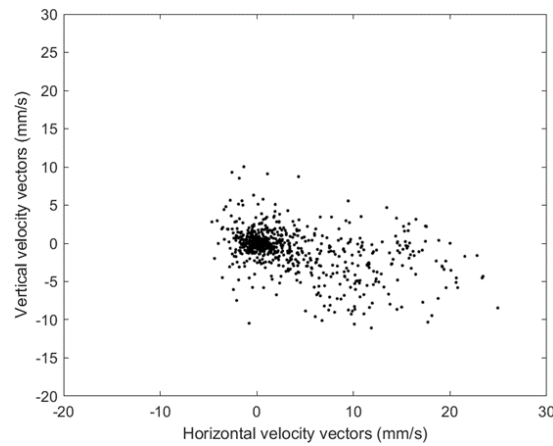


Figure 4-5 velocity vector filed

4.3 Statistical approach for detection of slip lane

Slip line formation is necessary for slope failure, slip line represent minimum safety factor(Cubas et al., 2008). During slope collapse, slope movement can be separated in two parts, which is unstable and stable parts. In this research, two primary modes of failure of slope lying on bedding plane with lateral confinements and toe support were identified: Toe sliding, wherein the entire slope shifts along the bedding plane, and Thrust failure, characterized by the movement of the upper portion while the lower remains stationary(Pongsakorn & Thirapong, 2023).

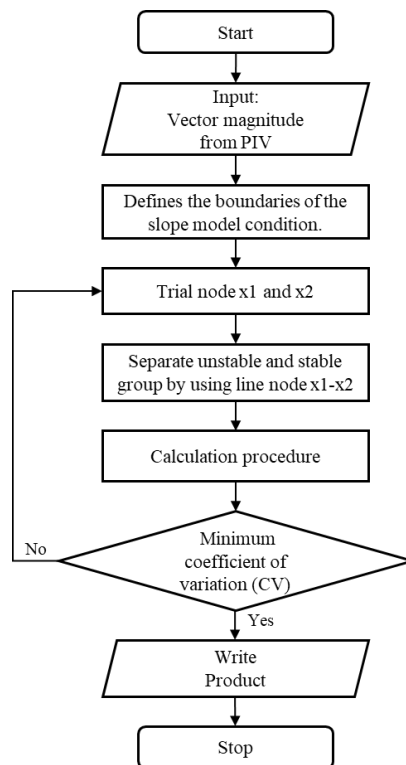


Figure 4-6 Flowchart of statistical of slip line detection

Slope failure occurrences during the experiment were documented through the utilization of a high-speed camera. To elucidate the behavioral dynamics of slope movement, Particle Image Velocimetry (PIV) was employed for the analysis of velocity vectors. Consequently, this article proposes a statistical methodology for determining the failure line. The coefficient of variation (CV) emerges as a pivotal statistical measure, delineating the dispersion of data points around the mean within a data series. It denotes the ratio of the standard deviation to the mean, facilitating the comparison of variation degrees across different data series, regardless of considerable differences in means. The delineation of the moving area, as illustrated in Figure 4-7, commences with PIV analysis to calculate velocity vectors and magnitude velocities from horizontal and vertical components, as delineated in Algorithm of Minimum coefficient. Geometric conditions are constrained by boundaries mirroring experimental conditions. Subsequently, trial nodes x1 and x2 are established along the upper and lower slope boundaries, respectively. The separation of two data groups by the line x1-x2 facilitates the computation of CV through the disparity in mean values and the amalgamation of standard deviations, as represented in Equations (b) and (c) respectively. Iteratively, the trial nodes x1-x2 are adjusted until achieving the minimum coefficient of variation (min_CV). Finally, the outcomes of the calculations are exported.

Algorithm Minimum coefficient of variation (CV)

1. Start
2. Input vector magnitude from PIV analysis
3. Defines the boundaries of the slope model condition.
4. Trial node x1 and x2
5. Separate unstable and stable group by using line x1-x2
6. Calculation procedure

Mean difference

$$\Delta\mu = \frac{1}{N_U} \sum_{k=1}^{N_U} V_k - \frac{1}{N_L} \sum_{k=1}^{N_L} V_k$$

Combination of standard deviation

$$\sigma_{UL} = \sqrt{\frac{\sum_{k=1}^{N_U} (V_k - \mu_U)^2 + \sum_{k=1}^{N_L} (V_k - \mu_L)^2}{N_U + N_L - 2}}$$

Coefficient of variation (CV)

$$CV = \frac{\sigma_{UL}}{\Delta\mu}$$

7. Find minimum CV
8. Repeat steps 4 to 7
9. Display and Print product
10. Stop

Figure 4-7 provides an illustrative example of the calculation process for test A2, demonstrating the velocity vector field obtained through Particle Image Velocimetry (PIV) analysis. The analysis depicts notable movement from the top to the middle of the slope, with comparatively lesser movement observed from the middle to the bottom. Initial coordinates for x_1 and x_2 are set at 0 and -30 respectively, while the final coordinates are adjusted to $x_1 = 260$ and $x_2 = 230$. The resulting minimum coefficient of variation ($\min_CV$) corresponds to $x_1 = 160$ and $x_2 = 46$.

A slip line is then delineated between nodes x_1 and x_2 , indicating two distinct areas: an unstable upper region and a stable lower region. This exemplifies the utility of the $\min_CV$ method in determining the slip line between these contrasting areas. It's important to note that the example presented here represents a single calculation for a specific interval of time, providing insight into the dynamics of slope behavior during that period.

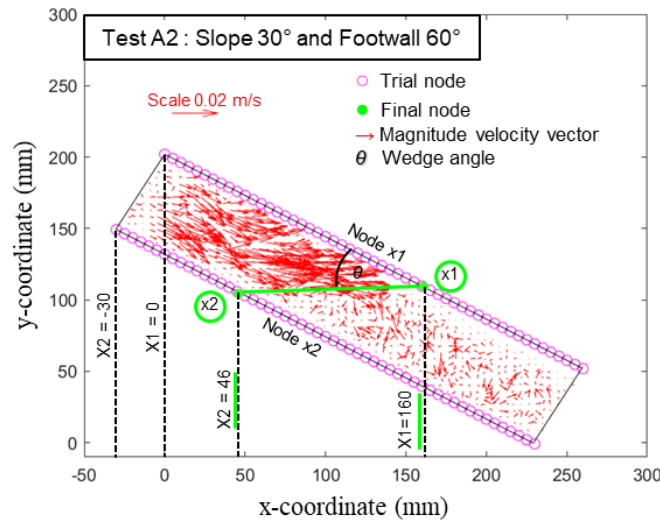


Figure 4-7 Calculation process of coefficient of variation

4.4 Calculation of failure time by minimum coefficient of variation

The purpose of the minimum CV method is to determine the optimal minimum CV value at each stage of the experimental process. This method is particularly useful in identifying the failure time in the context of the failure process. In this regard, the time of minimum coefficient of variation (t_c) serves as a proxy for the failure time, as it represents the peak disparity between two distinct areas.

Figure 4-8 provides a visual representation of the t_c values for the 12 tests, offering insights into the temporal evolution of slope instability. Additionally, Figure 4-9 presents the velocity vector field resulting from the optimal minimum CV calculation. Notably, Tests A1, A2, A3, B, C, E1, E2, F, G1, and G2 exhibited thrust failure during the minimum CV time, as indicated by the significant movement observed. However, Tests

D and H displayed toe sliding failures, characterized by slip lines closely aligned with the toe of the slope. Figure 4-10 shows parameters that contribute min CV calculations. Due to thrust failure, the slope can be divided into two parts: the upper part, representing the unstable, moving section of the slope, and the lower part, which remains stable. Consequently, the mean velocity and standard deviation of the upper part significantly increase during slope failure. Although the time of the minimum coefficient of variation (CV) has the potential to detect the initial failure slip line, this method exhibits some inconsistency when the failure slip line changes due to surcharge movement, as shown in Figure 4-11 to 4-22, owing to the inclusion of minimal velocity vectors above the surcharge in the minimum CV calculation.

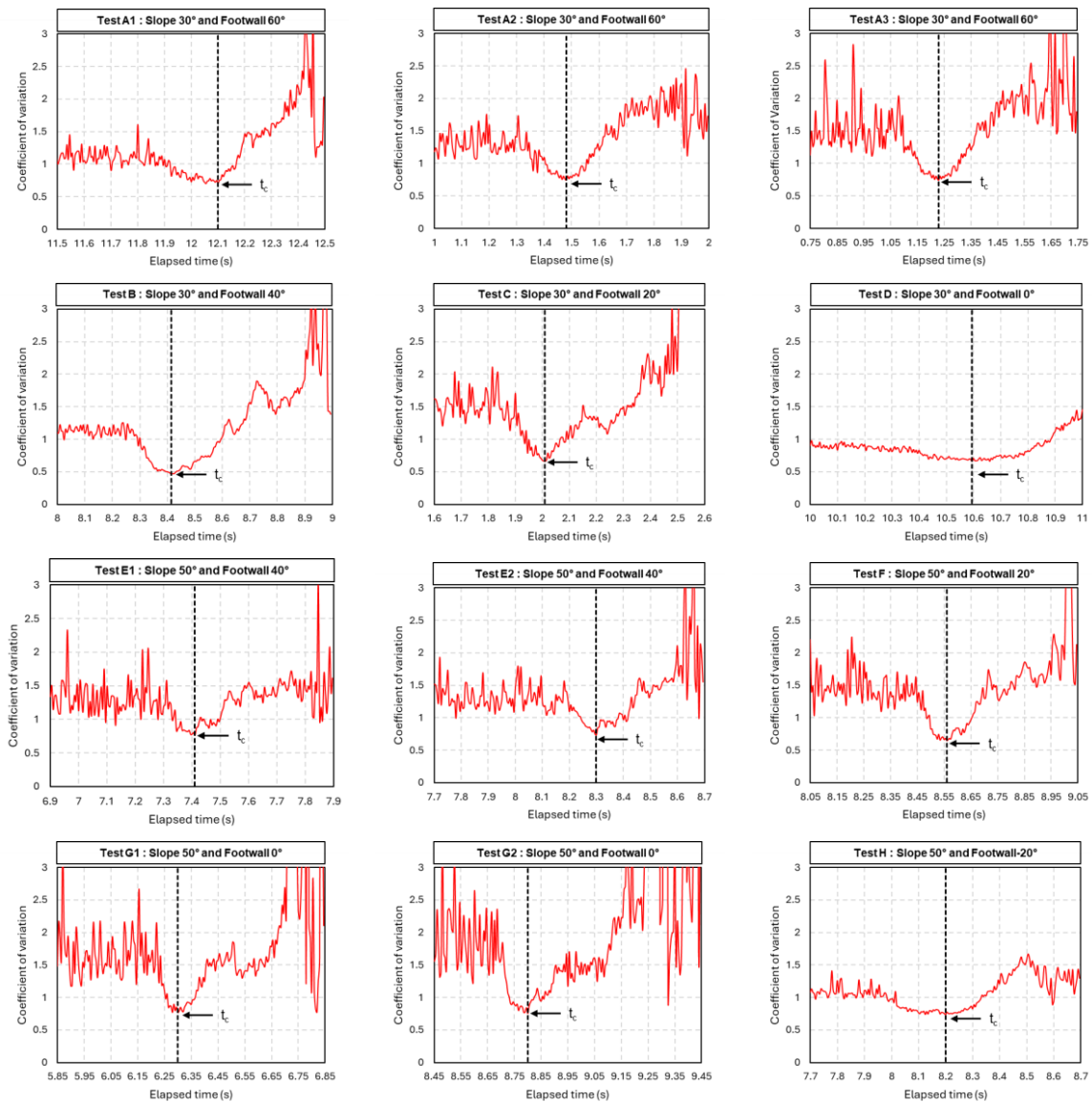


Figure 4-8 Time of Minimum CV

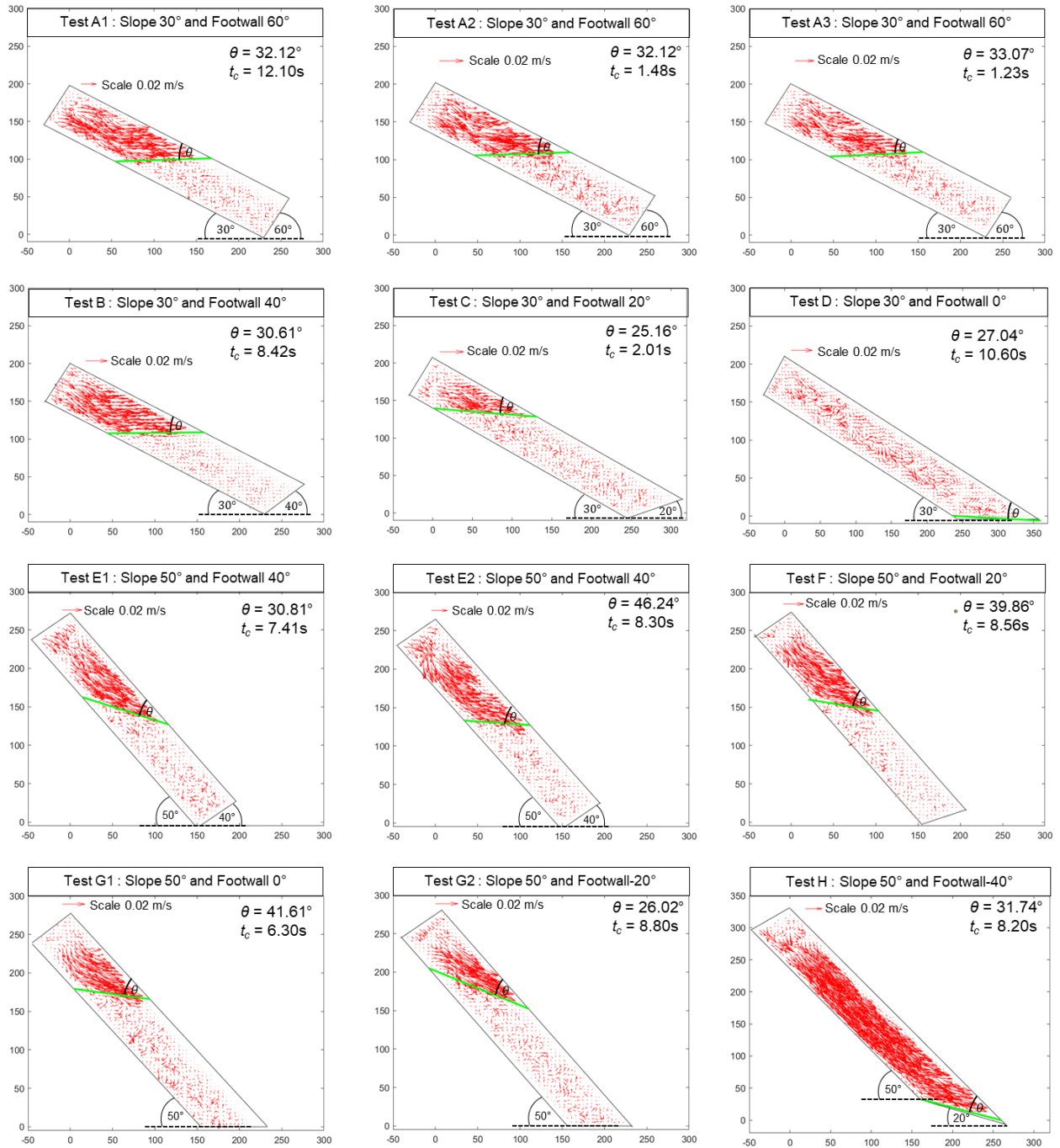


Figure 4-9 The result of slip line detection from Minimum CV

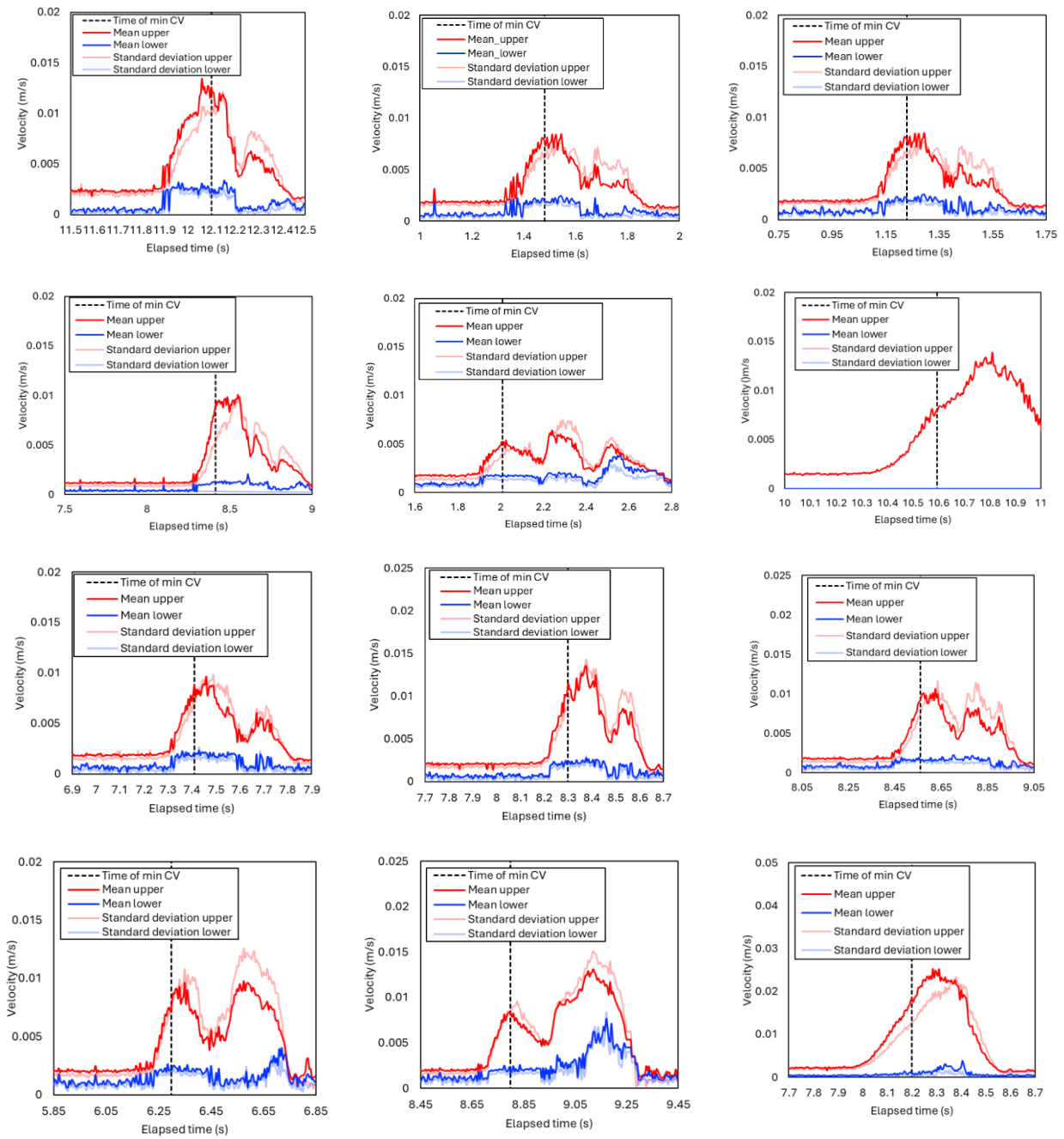


Figure 4-10 Correlation between time of minimum CV, mean velocity (upper and lower) and standard deviation (upper and lower)

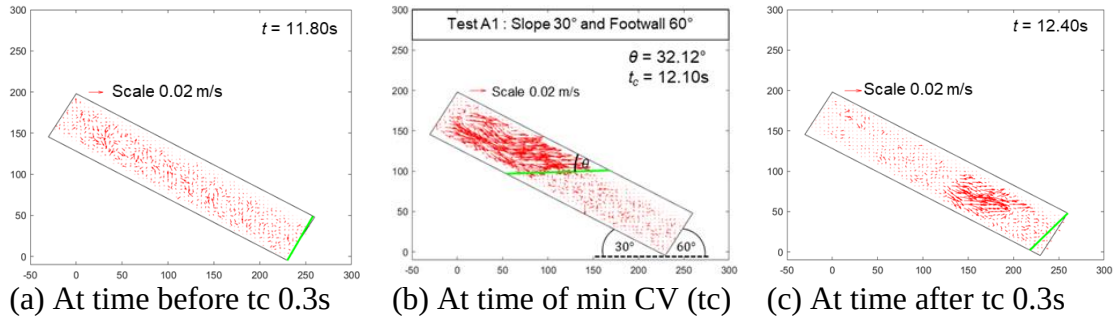


Figure 4-11 Snapshots of the velocity vector distribution for test A1

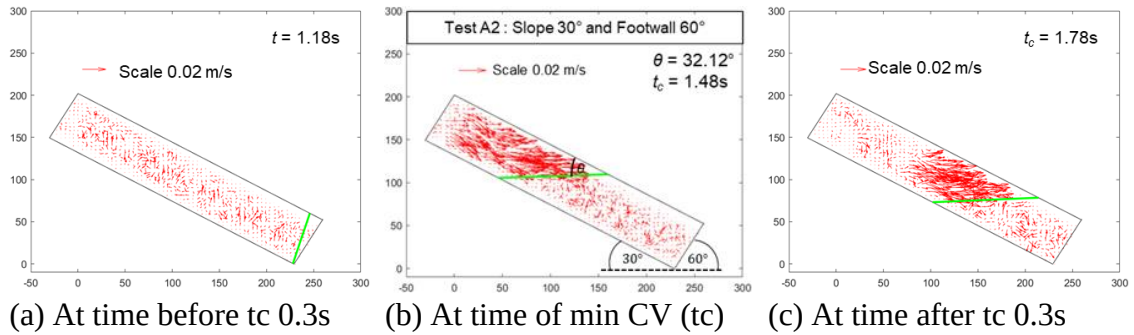


Figure 4-12 Snapshots of the velocity vector distribution for test A2

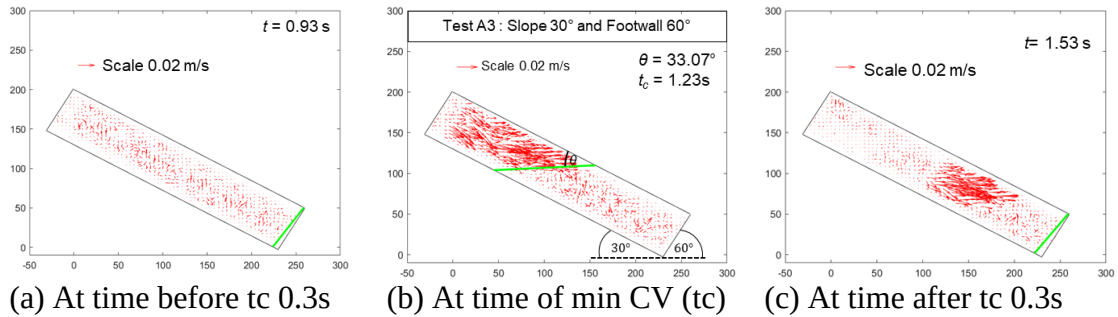


Figure 4-13 Snapshots of the velocity vector distribution for test A3

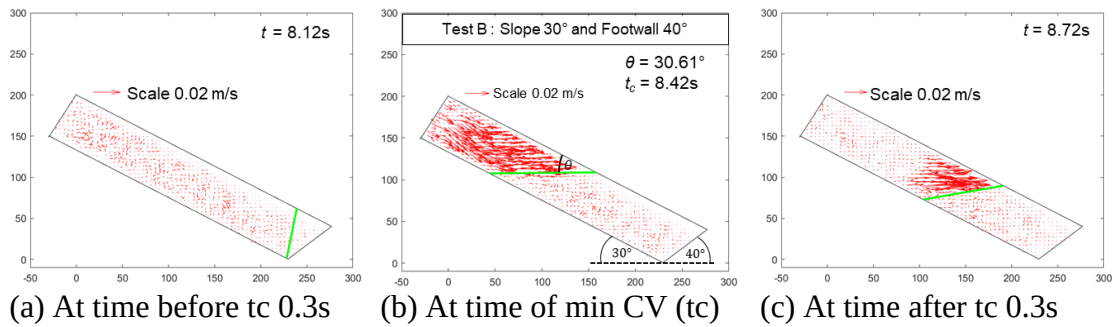


Figure 4-14 Snapshots of the velocity vector distribution for test B

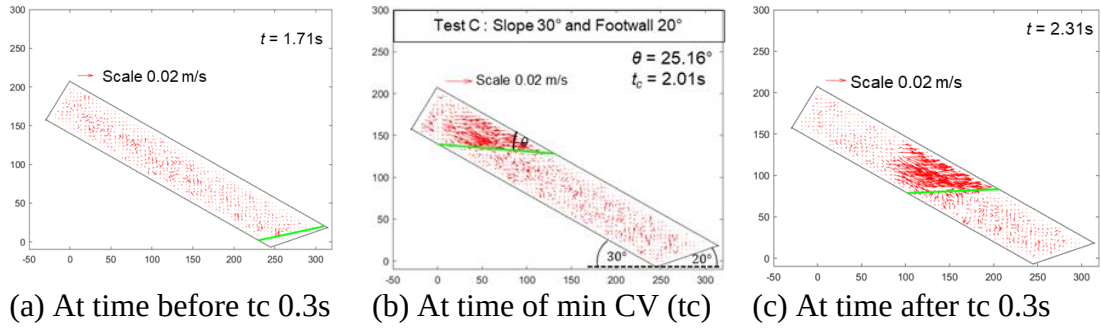


Figure 4-15 Snapshots of the velocity vector distribution for test C

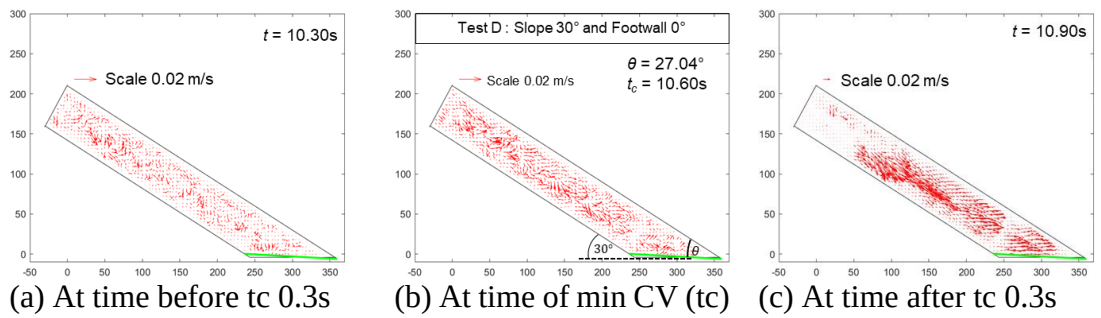


Figure 4-16 Snapshots of the velocity vector distribution for test D

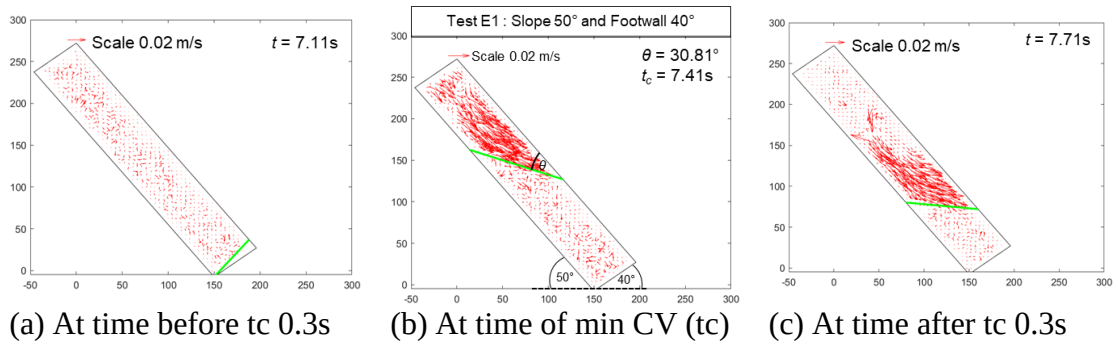


Figure 4-17 Snapshots of the velocity vector distribution for test E1

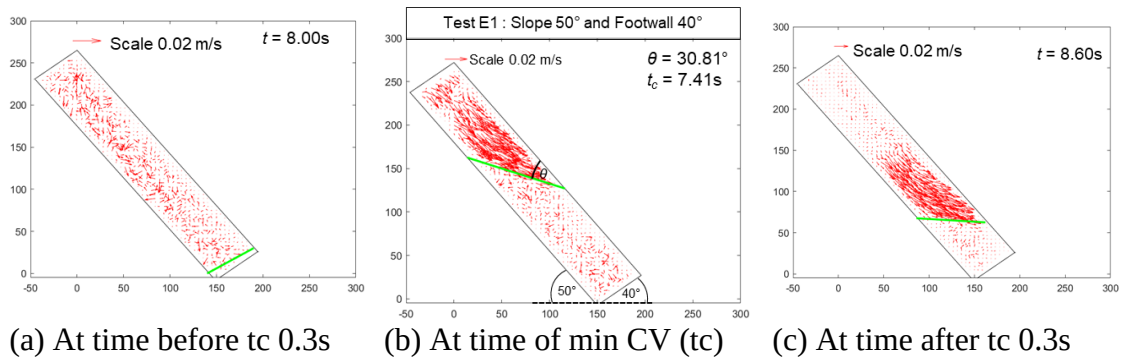


Figure 4-18 Snapshots of the velocity vector distribution for test E2

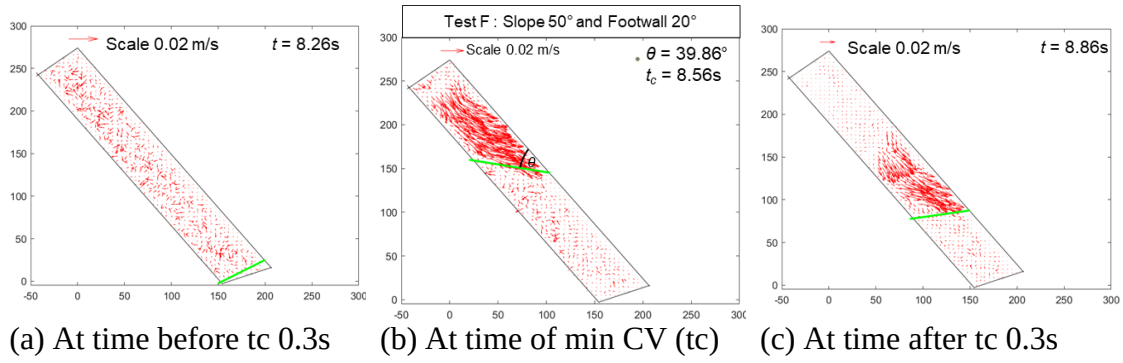


Figure 4-19 Snapshots of the velocity vector distribution for test F

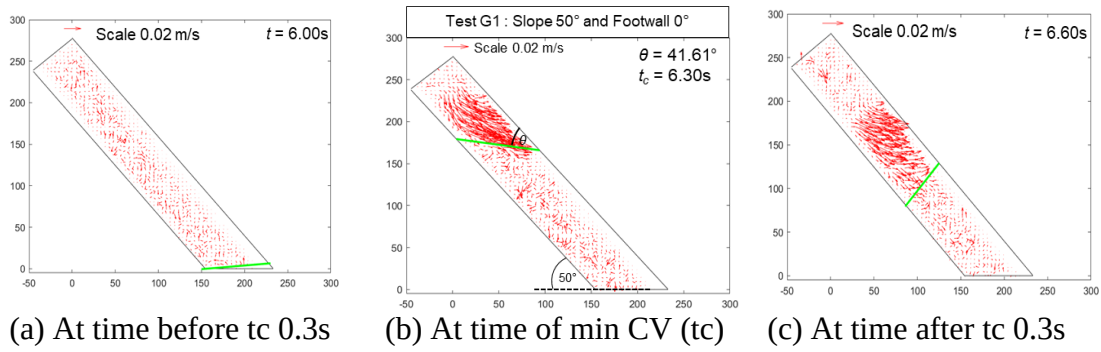


Figure 4-20 Snapshots of the velocity vector distribution for test G1

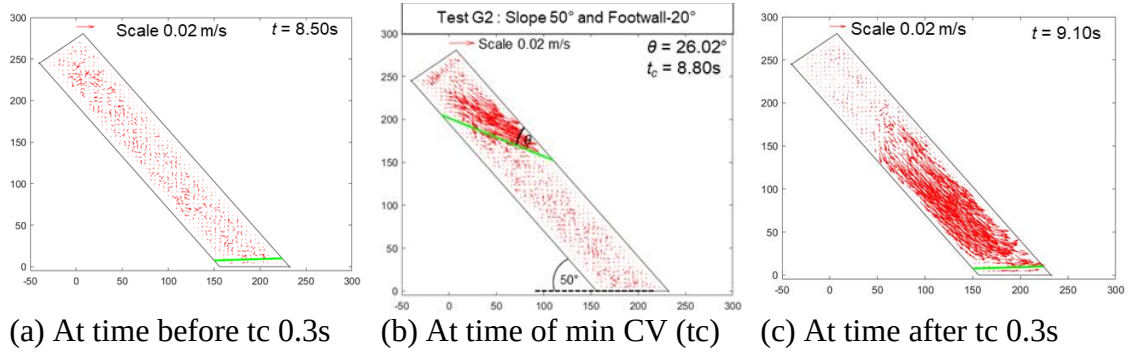


Figure 4-21 Snapshots of the velocity vector distribution for test G2

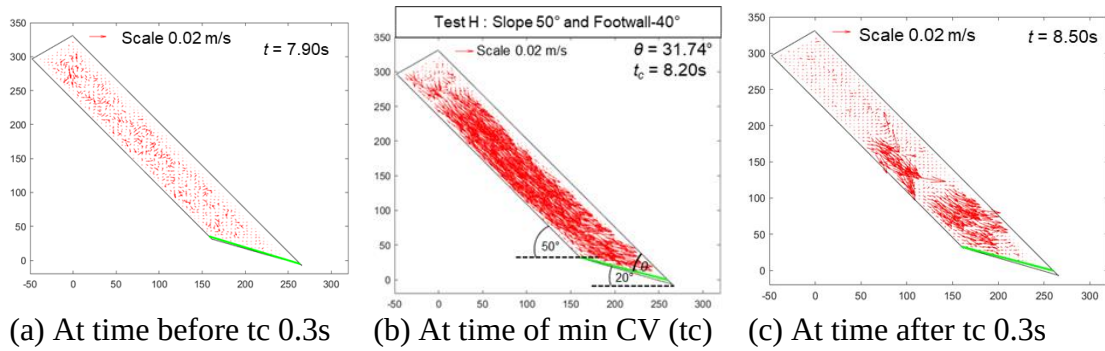


Figure 4-22 Snapshots of the velocity vector distribution for test H

Furthermore, Figure 4-23 provides a contour plot that demonstrates how the selection of nodes x1-x2 optimally facilitates the identification of the minimum CV. These graphical representations play a crucial role in comprehending the critical aspects of the failure process and provide valuable insights into slope behavior under various conditions. Table 4-2 illustrates the summation of all test results, offering a comprehensive overview of the findings across the slip line detection and time of minimum CV.

Wedge angle

Rankine (1857), a cornerstone principle in soil mechanics, operates on several fundamental assumptions to accurately model soil behavior under pressure. Firstly, it asserts the absence of friction between the backfill soil and the retaining wall, effectively assuming an angle of friction of zero. Secondly, it presupposes a vertical orientation for the retaining wall. Thirdly, it assumes a horizontal surface for the backfill soil.

Furthermore, the Rankine Theory posits that soil behavior depends on the movement of the retaining wall. If the wall advances into the soil, the soil adopts a passive state; conversely, if the wall moves outward away from the soil, the soil adopts an active state.

Upon reaching the failure state described by the Rankine Theory, passive failure zones emerge. These zones are characterized by specific attributes, often depicted in diagrams such as Figure 4-24(a). Additionally, the failure zone can be quantified by the wedge angle (θ), calculated using the formula $\theta = 45^\circ - \phi/2$, where ϕ represents the soil friction angle.

In this study, the soil slope was induced to fail by a surcharge cart, resulting in the failure zone being in a passive state. However, both the soil and the applied surcharge were inclined at angles of 30 and 50 degrees (α), respectively. The analysis of slip line formation using the minCV method revealed that the calculated wedge angle closely approximated the Rankine wedge angle ($45^\circ - 33.1/2$, 28.45). The wedge angles derived from the minimum coefficient of variation are presented in Table 4-2. It is worth emphasizing that in tests D and H, the failure zone was observed to be located at the toe support. This underscores the considerable influence of the toe angle (β) on slopes situated on a bedding plane.

The findings corroborate Pipatpongsa et al. (2023), wherein the influence of the reverse toe angle on the failure mode of slopes lying on bedding planes is discussed. Specifically, when the sum of the slope angle (α), reverse toe angle (β), and friction angle between toe support and material (ϕ_i) exceeds 90 degrees, as observed in Tests A1, A2, A3, B, C, E1, E2, F, G1, and G2, thrust failure tends to occur. Conversely, when the sum is less than 90 degrees, as seen in Tests D and H, toe sliding is more likely to occur. The alignment of the observed slip lines with these conditions further supports this correlation.

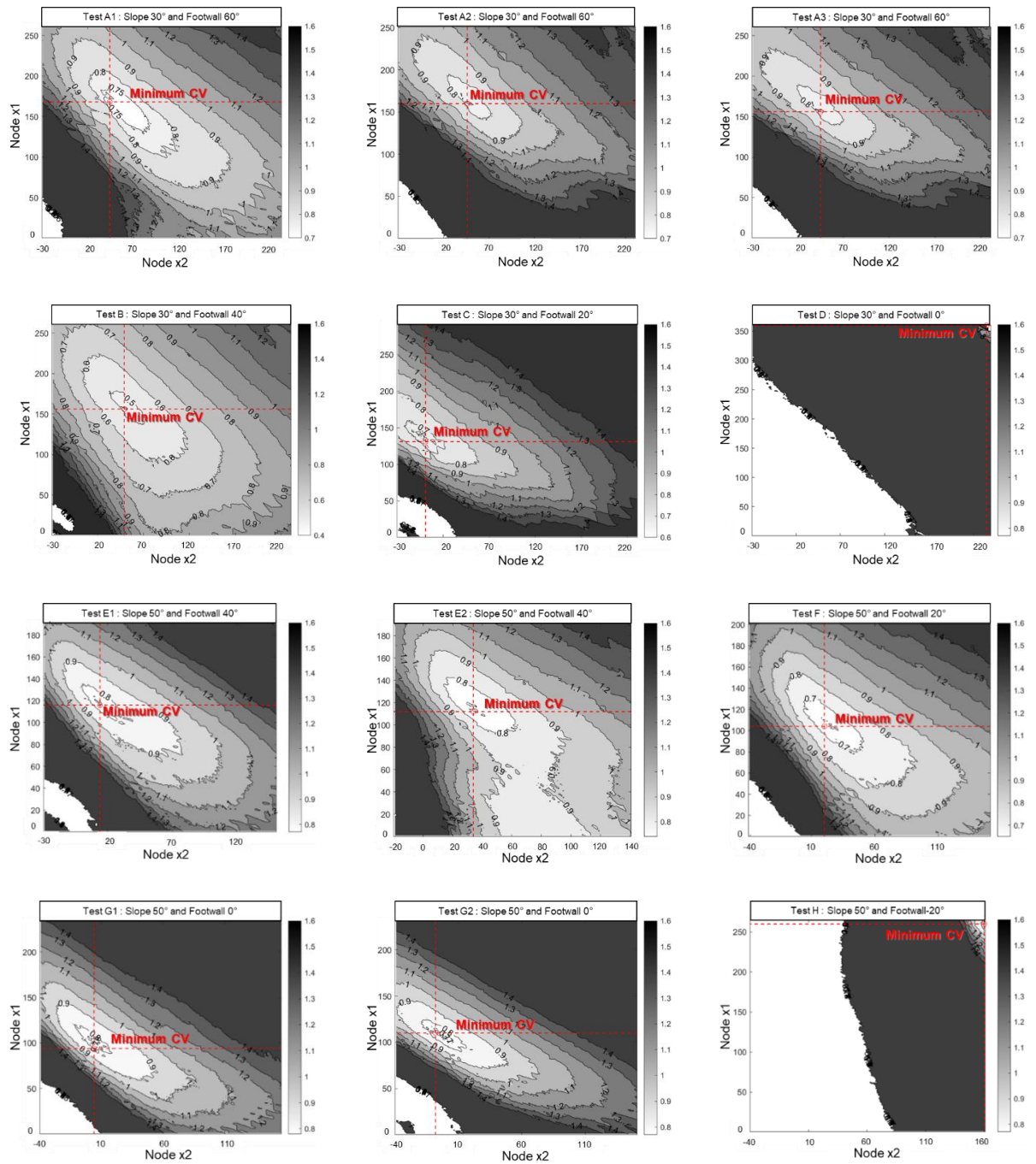


Figure 4-23 Contourplot of Minimum CV

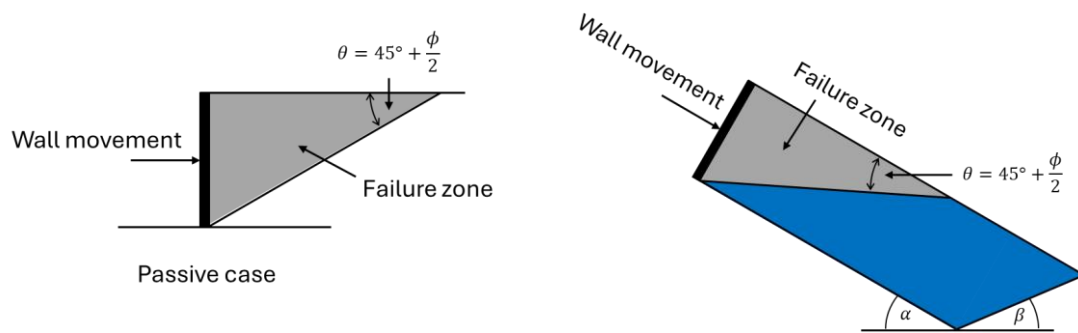


Figure 4-24 Rankine's passive wedge and conceptual of dip slope slip line

Table 4-2 Summation of statistical calculation

Test №	Slope angle α (°)	Toe angle β (°)	Failure mode	Wedge angle θ (°)	x1(mm)	x2(mm)	t_c (s)
A1	30	60	Fore-thrust	32.12	168	54	12.10
A2			Fore-thrust	32.11	160	46	1.48
A3			Fore-thrust	33.07	156	45	1.23
B		40	Fore-thrust	30.61	156	49	8.42
C		20	Fore-thrust	25.16	131	1	2.01
D	50	0	Toe	27.05	360	236	10.60
E1		40	Fore-thrust	30.81	116	14	7.41
E2			Fore-thrust	46.24	112	34	8.30
F		20	Fore-thrust	39.88	104	20	8.56
G1		0	Fore-thrust	41.61	94	4	6.30
G2		0	Fore-thrust	26.01	110	-8	8.80
H		-20	Toe	31.74	260	160	8.20

4.5 Inverse velocity method

Various monitoring techniques have been explored in practice to evaluate the stability of soil slopes and serve as early warning systems (Chen et al., 2018). These techniques encompass different monitoring objectives, such as accumulated displacement, intensity of rainfall, tilting angle, or groundwater level. Among these, the rate of displacement has been considered a crucial threshold for warning, as introduced by Saito (1961) and Saito (1965).

In both experimental and practical field scenarios, inverse velocity is widely employed to predict failure time (Fukuzono, 1985). In the experimental procedures conducted in this research, the slope was activated by applying a surcharge. As soon as a

sufficient amount of surcharge was added, the slope began to slide, marking the mobilized time (t_m). Subsequently, when the slope experienced significant movement and an increase in velocity, it reached the failure time (t_f).

Furthermore, PIV analysis and $\min_CV$ were utilized to provide velocity vectors and identify the moving area, which can be represented using a slip line. The average of velocity vectors within the moving area was then used to apply the inverse velocity method. Consequently, the inverse velocity method was employed in this research to determine the mobilized time (t_m) and the predicted time of failure (t'_c).

Moreover, the forecasted failure time was validated by cross-referencing it with the time of minimum coefficient of variation (t_c). In Figure 4-25(a), the region of interest utilized for inverse velocity computation is delineated. To ascertain the mobilized time (t_m) and predicted time of failure (t'_c), bilinear regression was employed, as depicted in Figure 4-26. Each data point in the graph represents the average velocity within the moving area. A maximum inverse velocity indicates slope stability, while an decreasing inverse velocity suggests slope instability. The bilinear regression method generates two lines: the first one signifies minimal movement leading to the mobilized time, and the second one denotes the time when the slope undergoes substantial movement. Additionally, the predicted failure time can be approximated by identifying the intercept of the second line with the x-axis.

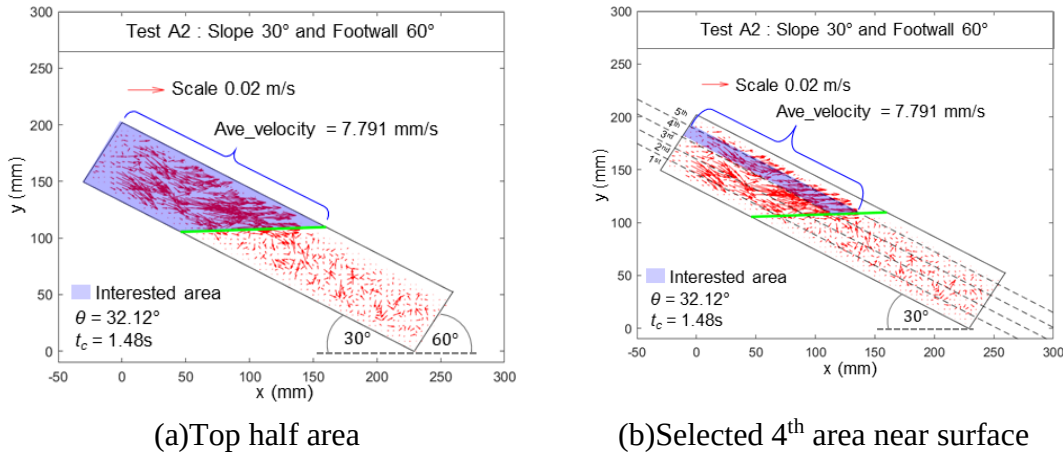


Figure 4-25 selected area for inverse velocity analysis

In this study, PIV analysis and inverse velocity were initially applied to the side view of the slope lying on the bedding plane. However, considering that slope movement is typically detectable only at the surface during real-site measurements, a practical approach was adopted. The slope was divided into 5 layers, with a specific focus on selecting the layer closest to the surface for analysis. Although the 5th layer seemed logical for this purpose, it was observed that this layer exhibited minimal movement of vectors. This lack of movement was likely due to shadow restrictions during the experiment, as illustrated in Figure 4-25(b).

Consequently, the 4th layer was selected for further analysis instead of the 5th layer. The average velocity of the 4th layer was then utilized for the inverse velocity method and for calculating the mobilized time (t_m) and predicted failure time (t'_s). This adjustment ensured that the analysis was based on the layer that best represented the surface movement of the slope, leading to more accurate results and interpretations.

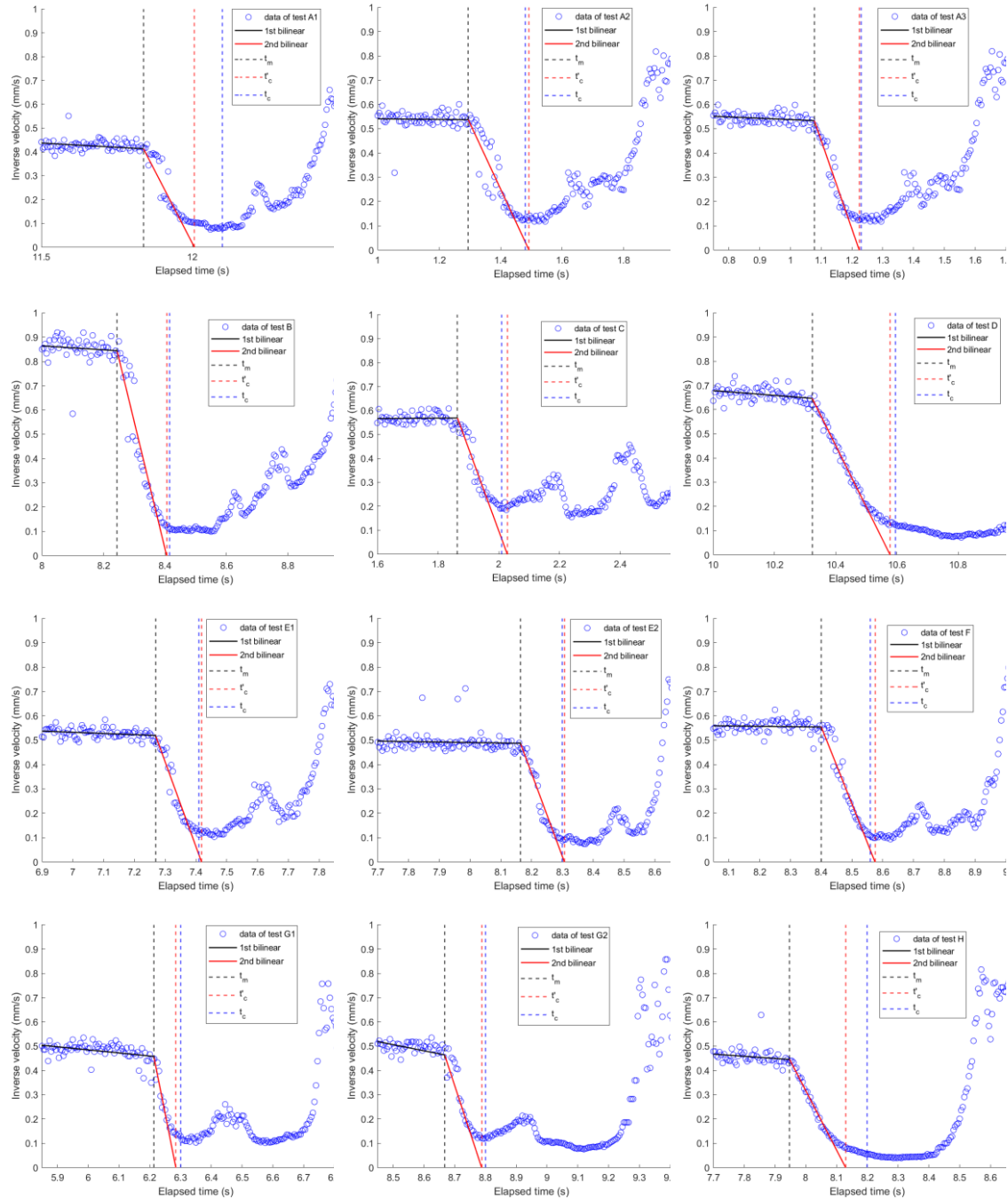


Figure 4-26 Inverse velocity and bilinear regression based on moving area by minimum CV

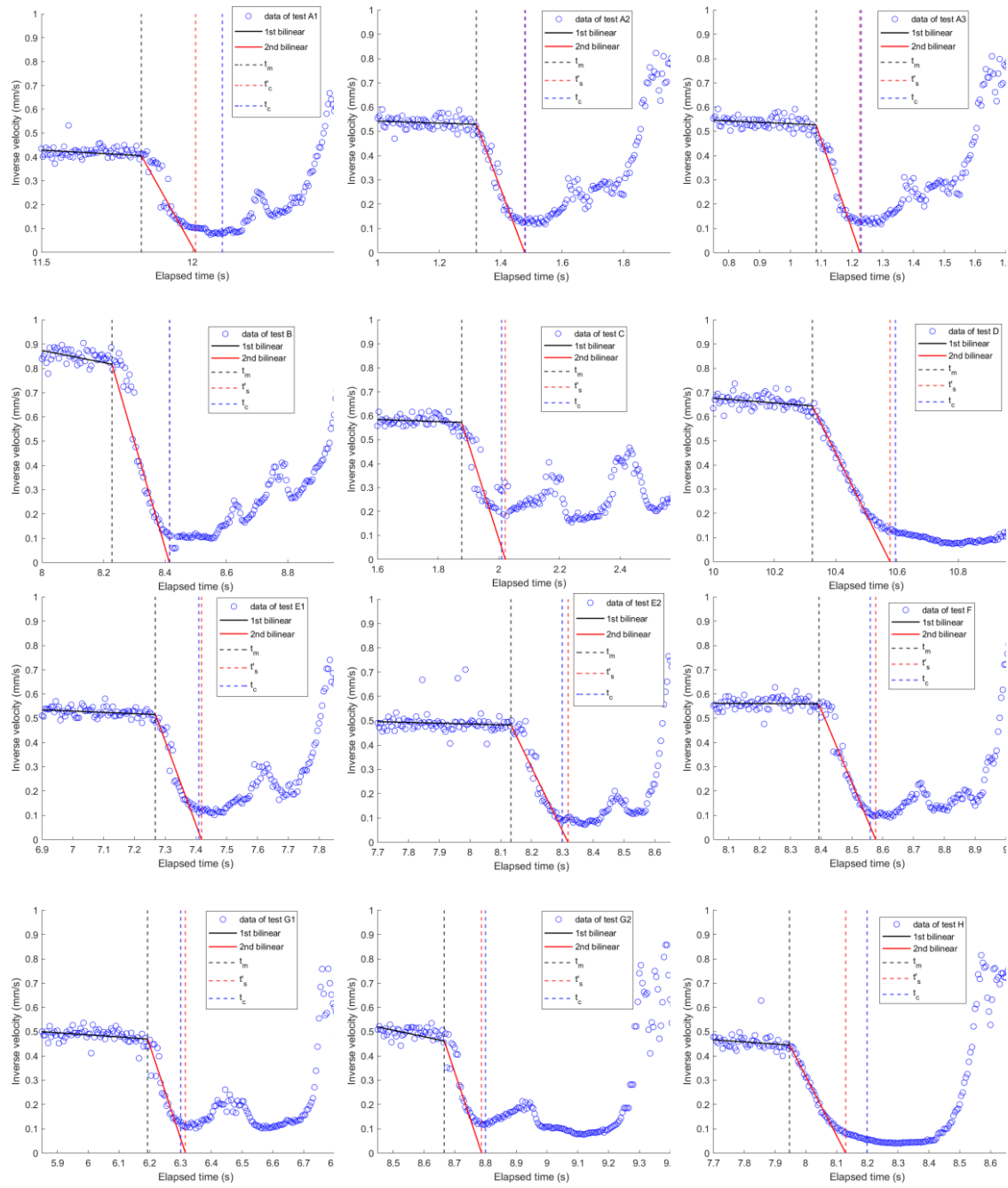


Figure 4-27 Inverse velocity and bilinear regression based on selected 4th area by minimum CV

Mobilized time based on results from moving area and selected 4th area are listed in Table 4-3.

Table 4-3 Summation of mobilize time and failure time

Test no.	$t_{m1}(s)$	$t_{m2}(s)$	$t_c(s)$	$t'_c(s)$	$t'_s(s)$
A1	11.838	11.831	12.100	12.007	12.011
A2	1.295	1.321	1.480	1.493	1.478
A3	1.078	1.083	1.230	1.224	1.226
B	8.244	8.227	8.415	8.406	8.415
C	1.864	1.878	2.010	2.028	2.022
D	10.323	10.323	10.595	10.578	10.578
E1	7.269	7.268	7.410	7.418	7.418
E2	8.164	8.133	8.300	8.307	8.318
F	8.401	8.392	8.560	8.575	8.578
G1	6.213	6.193	6.300	6.285	6.316
G2	8.667	8.666	8.800	8.788	8.788
H	7.948	7.948	8.200	8.130	8.130

where

- t_c Time of minimum COV (Real failure time)
- t_{m1} Mobilized time based on moving area of inverse velocity
- t_{m2} Mobilized time based on selected area(4th) of inverse velocity
- t'_c Predicted time by using bilinear regression from inverse velocity (Predicted failure time)
- t'_s Near surface predicted time by using bilinear regression from inverse velocity (Predicted failure time)

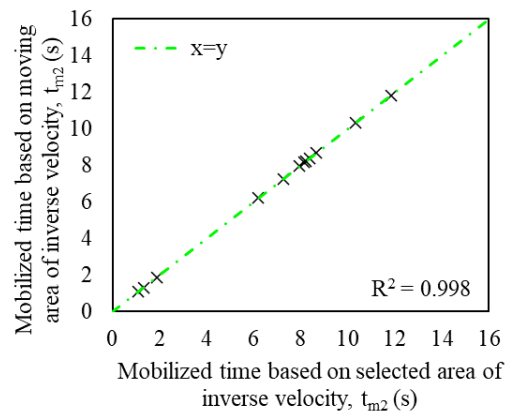


Figure 4-28 Correlation between mobilized time of selected area and moving area

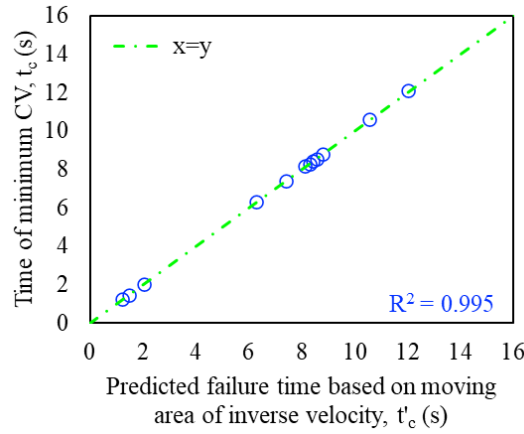


Figure 4-29 Correlation between time of failure based on inverse velocity and minimum CV

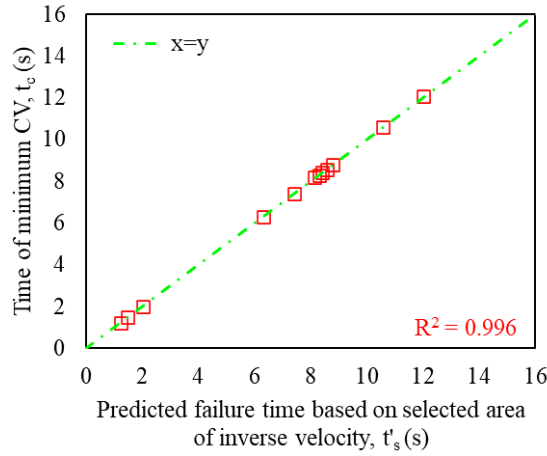


Figure 4-30 Correlation between time of failure based on selected area on inverse velocity and minimum CV

The findings in Figure 4-29 underscore a strong alignment between the time of minimum coefficient of variation (t_c) and the predicted failure time (t'_c) determined through bilinear regression of inverse velocity. High coefficients of determination (R^2), peaking at 0.995, validate this correlation, affirming t_c ' reliability as a proxy for the time of failure (t_f).

Moreover, focusing on the 4th layer of the slope yielded promising outcomes, with R^2 values up to 0.996 compared to t_c , shown in Figure 4-30. This demonstrates the effectiveness of this layer for analysis, bolstering confidence in its practical application and alignment with site measurements.

Overall, these results provide robust confirmation of the utility of the time of minimum CV (t_c) as an indicator of slope failure, as well as the effectiveness of selecting

the 4th layer of the slope for analysis, highlighting the potential for practical application in real-world scenarios.

4.6 Summary

In Chapter 4, the failure mechanisms with various reverse toe angles were observed and analyzed using Particle Image Velocimetry (PIV). The outcomes were derived from 1g experimental tests encompassing 12 distinct cases. A statistical approach was introduced to detect the formation of slip lines during the failure process. Additionally, the inverse velocity method was employed to predict the time of failure.

- The failure mechanism of slopes can be categorized into two distinct modes: toe sliding and fore-thrust failure. The presence of a reverse toe angle significantly influences the occurrence of these failure modes, as corroborated by the results of Particle Image Velocimetry (PIV) analysis. In a series of slopes with a 30-degree inclination, toe sliding was observed when the reverse toe angle was 0 degrees. Conversely, for slopes inclined at 30 degrees with reverse toe angles ranging from 60 to 20 degrees, fore-thrust failure occurred. Similarly, in slopes inclined at 50 degrees, toe sliding occurred when the reverse toe angle was -20 degrees, while fore-thrust failure was evident for slopes with reverse toe angles ranging from 40 to 0 degrees.
- The Minimum Coefficient of Variation (MIN_CV) method is newly designed to detect the formation of slip during slope failure. The position of the slip line is contingent upon the reverse toe angle and is directly associated with the failure mechanism. The Rankine passive theory was employed to calculate the wedge angle ($\theta = \pi/4 - \phi/2 = 29^\circ$). The computed wedge angle from the MIN_CV slip line method averages at 33.3 degrees, which closely aligns with Rankine's passive wedge theory.
- In parallel, the inverse velocity method was introduced to predict the time of failure. Bilinear regression was utilized to determine the time at which the slope mobilizes and to predict the time of failure. However, the time of minimum CV represents the moment of greatest discrepancy in velocity between two parts and can thus serve as an indicator of the time of failure. The results obtained from the inverse velocity method closely approximate the predicted time of failure.

Chapter 5 Centrifuge Model of Dip Slope on Impermeable Bedding Plane with Groundwater Infiltration and Passive Thrust Induced Instability

In this chapter, a series of centrifuge tests were conducted to study slope failure subjected to groundwater seepage on an impermeable bedding plane, using damp Hiroshima sand. A surcharge load cart acted as a moving boundary, triggering the loading in these experiments. The slope geometries were designed based on the findings from Chapter 4. Due to the effect of the reverse toe angle on the failure mechanism of dip slopes, two series of centrifugal modeling were performed: one with a slope angle of 30 degrees and a reverse toe angle of 0 degrees, and the other with a slope angle of 30 degrees and a reverse toe angle of 15 degrees. These two slope geometries exhibit different modes of failure, identified in Chapter 4 as toe sliding and thrust failure, respectively.

Theoretical calculations of slope stability are provided in Section 5.1. The experimental setup and methodology are explained in Sections 5.2 and 5.3, respectively. In Section 5.4, experimental results are reported. PIV analysis was employed to confirm the mode of failure in Section 5.4.1. Considering the importance of the reduction ratio for slope stability calculations, the reduction ratio was calculated and reported in Section 5.6. In Section 5.7, the factor of safety was calculated and compared based on predictions and experimental results. Additionally, back-calculation methods were used to enhance the accuracy of the calculations by comparing them with experimental results. Finally, the findings obtained in this chapter are summarized in Section 5.8.

5.1 Theoretical calculations of slope lying on bedding plane with lateral and toe supports.

In this study, a physical model of a slope lying on a bedding plane with lateral confinement and toe support was examined. The model, illustrated in Figure 5-1, comprised two lateral supports on the sides and various toe angles. Triggering of the slope model was achieved using a surcharge load cart, which served as a moving boundary to simulate the effect of a large weight or mass on the slope, with a particular focus on the region near the toe support. However, it was assumed that the sand block behaves as a rigid body. Under the application of the surcharge, the sand block should deform symmetrically.

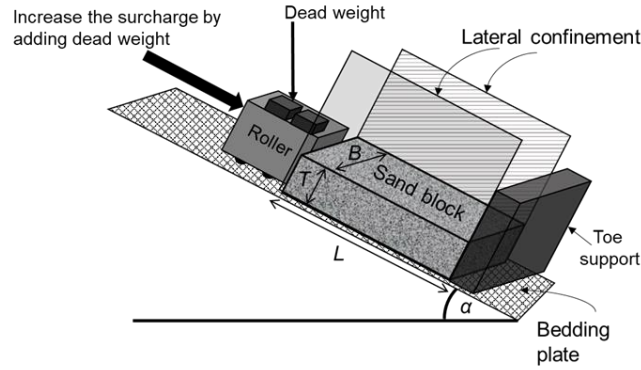


Figure 5-1 Idealization of slope lying on bedding plane with lateral confinements and toe support

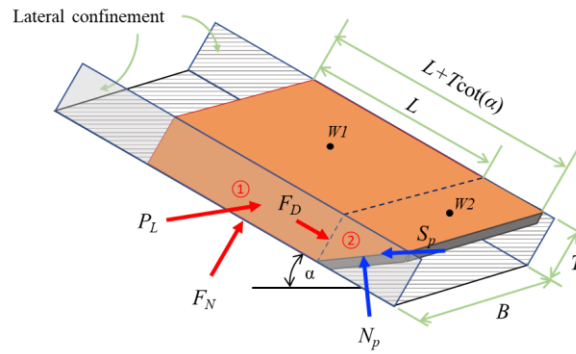


Figure 5-2 Diagram of force acting on slope model

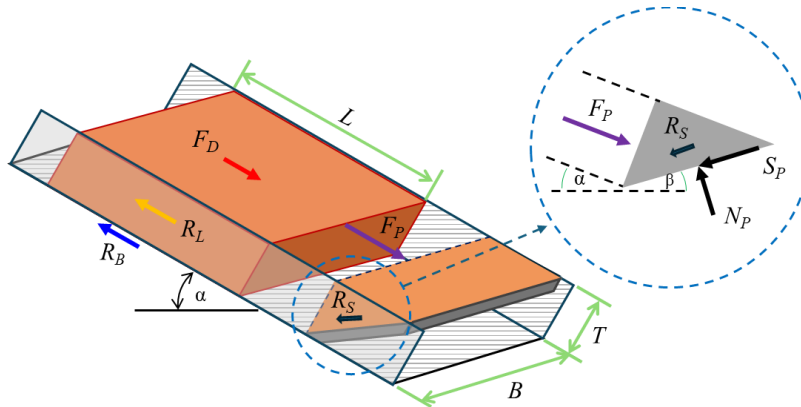


Figure 5-3 Diagram of acting force on separated slope and toe part

For the purpose of calculation, the slope was divided into two distinct parts, as depicted in Figure 5-2: a rectangular block, ① and a triangular block, ②. Parameters such as the length (L), breadth (B), thickness (T), slope angle (α), and toe angle (β) were considered in the analysis. Additionally, a surcharge weight (ΔW) contained within a cart and resting on rollers was incrementally added to the top of the slope model to induce instability.

This setup facilitated the investigation of the behavior and response of the slope under varying conditions and loading scenarios.

The gravitational force acting on the soil is represented by Eqs. (5-1) and (5-2). Furthermore, the constituent forces of the soil's weight (the normal force, F_N and the driving force, F_D) are determined by Eqs. (5-3) to (5-5). Additionally, surcharge weight ΔW is also added in F_D , where ϕ_r is the friction angle of rollers.

$$W_1 = \gamma BTL \quad (5-1)$$

$$W_2 = \frac{\gamma B T^2}{2} \cot(\alpha + \beta) \quad (5-2)$$

$$F_D = \gamma BTL \sin(\alpha) + \Delta W(\sin(\alpha) - \tan(\phi_r) \cdot \cos(\alpha)) \quad (5-3)$$

$$F_{N1} = W_1 \cos(\alpha) \quad (5-4)$$

$$F_{N2} = W_2 \quad (5-5)$$

Figure 5-3 illustrates the shear resistance on the bedding, lateral support, and toe support. The bedding shear resistance (R_B), derived from the Mohr-Coulomb failure criterion, is represented by Eq (5-6), utilizing the interface friction angle (ϕ_i) and adhesion (c_i).

$$R_B = F_{N1} \tan(\phi_i) + c_i LB \quad (5-6)$$

The lateral resistance for rectangular blocks is determined by the normal stress acting on the lateral support and the Mohr-Coulomb failure criterion. However, the normal force on the lateral support must be defined using Rankine's earth pressure theory. The shear strength reduction to define the developed lateral earth pressure (σ_L) at any thickness along the lateral support of the rectangular block is expressed as a function of thickness t , where K is the coefficient of lateral earth pressure, c is the cohesion of the slope material, and r_d is the reduction ratio.

$$\sigma_{N1}(t) = \frac{F_{N1}}{BL} \quad (5-7)$$

$$\sigma_L(t) = \sigma_{N1}(t)K(r_d) + 2r_dc\sqrt{K(r_d)} \quad (5-8)$$

Reduction ratio r_d , is defined as the ratio of τ_d to τ_f as shown in Eq. (5-9). Given that σ'_n is an arbitrary value, r_d concerning shear strength also equals r_d concerning friction and cohesion. The reduction ratio (r_d) serves as a tool to estimate whether the soil is in an active (-1) or passive (+1) state. Subsequently, r_d is utilized to compute K . For $r_d = -1$, K_a (active earth pressure, $\tan^2(\frac{\pi}{4} - \frac{\phi}{2})$) is expressed, while for $r_d = +1$, K_{pa} (passive earth pressure, $\tan^2(\frac{\pi}{4} + \frac{\phi}{2})$) is utilized.

$$r_d = \frac{\tau_d}{\tau_f} = \frac{\sigma'_n \tan \phi_d + c_d}{\sigma'_n \tan \phi + c} = \frac{\tan \phi_d}{\tan \phi} = \frac{c_d}{c} \quad (5-9)$$

$$K(r_d) = \tan^2\left(\frac{\pi}{4} + \frac{\tan^{-1}(r_d \tan \phi)}{2}\right) \quad (5-10)$$

Once r_d and K are determined, lateral shear resistance R_L can be defined by Mohr-Coulomb failure criterion, as shown in Eq. (5-11) where ϕ_s represents interface friction and c_s denotes adhesion between the slope and lateral support.

$$\begin{aligned} R_L &= \int_0^T (\sigma_L(t) \tan(\phi_s) + c_s) L dt \\ &= \left[\left(\frac{K(r_d)\gamma T}{2} \cos \alpha + 2cr_d \sqrt{K(r_d)} \right) \tan \phi_s + c_s \right] LT \end{aligned} \quad (5-11)$$

The triangular block will remain stationary if the driving force F_D is less than summation of R_B and $2R_L$ (2sides lateral supports). However, once $F_D > R_B + 2R_L$, remaining force (F_P) will initiate movement of the triangular block. F_P can be calculated using Eq. (5-12)

$$F_P = F_D - R_B - 2R_L \quad (5-12)$$

Therefore, the passive thrust limit, denoted as $F_P \leq P_T$, exerted on the virtual plane, is defined by the passive thrust P_T , which is determined by integrating σ_L across the thickness of the slope.

$$P_T = \int_0^T \sigma_L(t) B dt = \left(\frac{K(r_d)\gamma T}{2} \cos \alpha + 2cr_d \sqrt{K(r_d)} \right) BT \quad (5-13)$$

The lateral resistance (R_s) of the triangular block, determined akin to R_L , is expressed in Eq. (5-14). Notably, the direction of R_s opposes the sliding direction.

$$\begin{aligned} R_s &= \int_0^T (\sigma_L(t) \tan(\phi_s) + c_s) (T - t) \cot(\alpha + \beta) dt \\ &= \left[\left(\frac{K(r_d)\gamma T}{3} \cos \alpha + 2cr_d \sqrt{K(r_d)} \right) \tan \phi_s + c_s \right] \frac{T^2}{2} \cot(\alpha + \beta) \end{aligned} \quad (5-14)$$

The force acting on triangular block show in Figure 5-3. F_P is force trigger the triangular move. W_s is represent soil weight triangular block, R_s is lateral resistance force. At the toe bedding plane normal force reaction (N_P) and tangential reaction (S_P) are determined as Eqs. (5-15) and (5-16)

$$N_P = F_P \sin(\alpha + \beta) + W_2 \cos \beta \quad (5-15)$$

$$S_P = F_P \cos(\alpha + \beta) - W_2 \sin \beta - 2R_2 \quad (5-16)$$

At toe bedding, the resistance force can be calculated based on Mohr-Coulomb failure criterion. That means if S_P more than toe bedding resistance force, it have a potential to occur toe sliding. On the other hand, S_P less than resistance force, passive thrust failure can be occurred. Thus, S_P is limited by regrading resistance using following inequality, where ϕ_t is the interface friction angle, and c_t is the adhesion between the slope and toe support.

$$S_P \leq N_P \tan(\phi_t) + c_t BT \csc(\alpha + \beta) \quad (5-17)$$

To assess the limit of F_p on the virtual plane against sliding along the toe support, substituting Eqs. (5-15) and (5-16) into inequality (5-17) results in the following inequality after rearrangement:

$$F_p \cos(\alpha + \beta + \phi_t) \sec \phi_t \leq c_t BT \csc(\alpha + \beta) + 2R_s - W_2 \sin(\beta + \phi_t) \sec \phi_t \quad (5-18)$$

If $\cos(\alpha + \beta + \phi_t) \leq 0$ or $(\alpha + \beta) \geq \pi/2 - \phi_t$ then inequality (5-18) is valid because the right-hand side of the equation is always positive within a normal range as compression ensures $F_p > 0$. However, if $\cos(\alpha + \beta + \phi_t) \geq 0$ or $(\alpha + \beta) \leq \pi/2 - \phi_t$, then the limit $F_p \leq P_s$ can be thoroughly derived from inequality (5-16), where the sliding thrust P_s is determined by Eq. (5-19) through algebraic manipulation.

$$P_s = \frac{(c_t BT \csc(\alpha + \beta) + 2R_s) \cos(\phi_t) + W_2 \sin(\beta + \phi_t)}{\cos(\alpha + \beta + \phi_t)} \quad (5-19)$$

It should be noted that $P_s > 0$ indicates the threshold for reverse toe sliding, while $P_s < 0$ suggests no sliding occurs on the toe support and R_s acts in the opposite direction, as shown in Figure 5-3. The compressive thrust F_p must not surpass the limits set by either the passive thrust P_T or the sliding thrust P_s , provided that $FS > 0$. Consequently, thrust resistance R_p is defined as P_T if P_s is negative; otherwise, it should be the positive minimum between P_s and P_T , as expressed in the following conditional equation:

$$R_p = \begin{cases} P_T & \text{if } P_s < 0 \\ \min\{P_s, P_T\} & \text{otherwise} \end{cases} \quad (5-20)$$

Then normal stress and shear stress underneath the triangular block are calculated by N_p and S_p based on the contact area with the toe support, as expressed in following equation:

$$\sigma = \frac{R_p \sin(\beta + \phi_t) + W_2 \cos \beta}{BT \csc(\alpha + \beta)} \quad (5-21)$$

$$\tau = \frac{R_p \cos(\alpha + \beta) - W_2 \sin \beta - 2R_s}{BT \csc(\alpha + \beta)} \quad (5-22)$$

The total resistance is composed of three components: bedding resistance (R_B), lateral resistance (R_L) from both side and toe resistance (R_p). This relationship is expressed by the following equation.

$$R_T = R_p + R_B + 2R_L \quad (5-23)$$

Therefore, slope stability can be calculated and represented as factor of safety as determined by Eq. (5-24)

$$FS = \frac{R_T}{F_D} \quad (5-24)$$

Moreover, surcharge weight can be predicted to reach $FS = 1.00$ by substitute driving force and total resistance as Eq. (5-25)

$$\Delta W = \frac{R_p + R_B + 2R_L - W_1 \sin \alpha}{\sin \alpha - \tan \phi_t \cos \alpha} \quad (5-25)$$

The primary focus of this research is groundwater seepage. Therefore, the water profile is adopted from the work of Take et al. (2004). Water is introduced at the crest of the slope, flows through the soil as parallel seepage, and ultimately drains out at the toe part (see Figure 5-4).

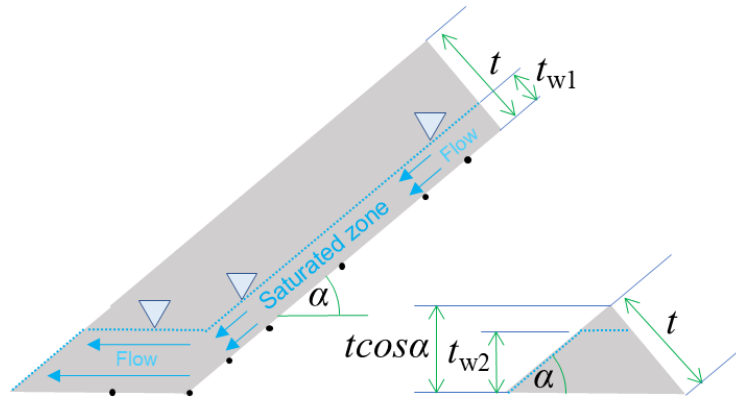


Figure 5-4 Groundwater seepage on slope lying on bedding plane

During water supplementation, the soil can be divided into saturated and unsaturated zones. Consequently, the unit weight of the soil varies with the height of the seepage level, as described in Eq. (5-26). The parameter n_w represents the water ratio, which can be determined by pore-water pressure according to Eq. (5-27). The modification of the soil weight for two blocks is expressed by Eqs. (5-28) and (5-29).

$$\gamma(n_w) = \gamma(1 - n_w) + \gamma_{sat} n_w \quad (5-26)$$

$$U_w = \gamma_w n_w T \cos(\alpha) \quad (5-27)$$

$$W_1 = \gamma(n_{w1}) B T L \quad (5-28)$$

$$W_2 = \frac{\gamma(n_{w2}) B T^2}{2} \cot(\alpha + \beta) \quad (5-29)$$

Equivalent interface adhesion subjected to groundwater seepage.

$$c' = c - U_w \tan \phi \quad (5-30)$$

5.2 Experimental setup and preparation of centrifuge physical model

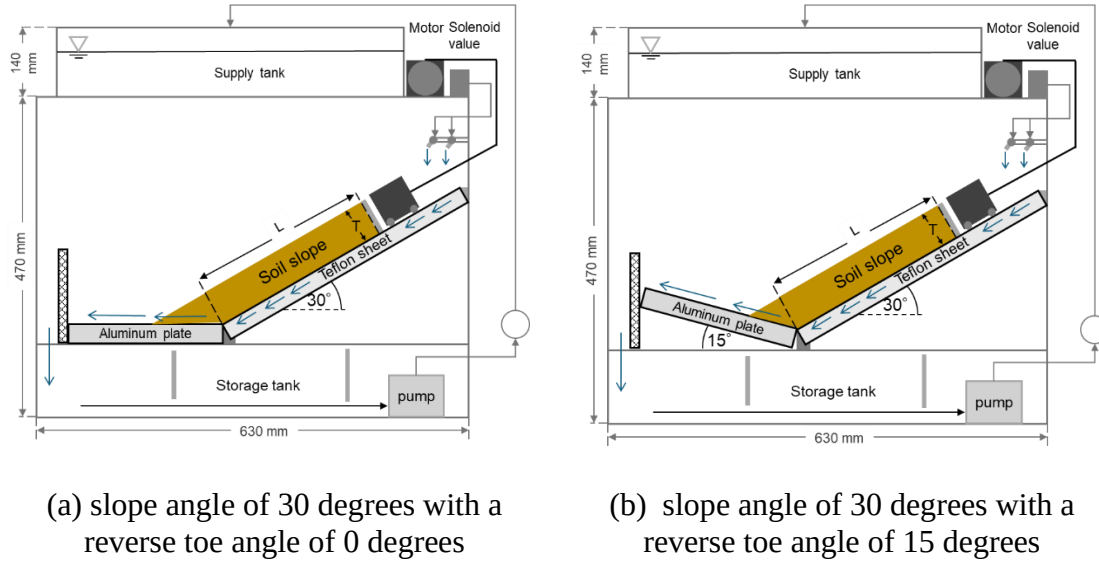


Figure 5-5 schematic diagram of triggering for centrifuge model

The experiments comprised six cases, classified into two series. The first series entailed a slope angle of 30 degrees with a reverse toe angle of 0 degrees (refer to Figure 5-5(a)), while the second series involved a slope angle of 30 degrees with a reverse toe angle of 15 degrees (refer to Figure 5-5(b)). During the experimental procedure, slope movement was induced by two factors: the surcharge weight, acting as a moving boundary to exert pressure on the slope in the model test, and watering, simulating groundwater seepage. As depicted in the schematic diagram in Figure 5-5, the surcharge load cart was controlled by a motor, while water supplementation was regulated by a solenoid valve connected to the top supply tank.

The experimental material utilized in this study was Hiroshima sand, sourced from Kure Shi, Hiroshima prefecture. Initial processing involved sieving the sand to a diameter of 1mm. Subsequently, the sand was adjusted to a water content of 8% before compaction. Compaction was conducted to achieve a compaction degree of 90%. The basic properties of the Hiroshima sand are summarized in Table 5-1.

To determine the interface shear strength, tilting tests were conducted. Given the potential for soil sliding along the bedding plane, as well as the presence of lateral and toe supports, Teflon sheets were employed as the bedding plane, while aluminum materials were utilized for the lateral and toe supports in the experimental setup.

Table 5-1 Basic properties of Hiroshima sand

Dry maximum density (ρ_{dmax})	1.747 g/cm ³
Water content (w)	8.0%
Degree of compaction (D_c)	90%
Coefficient of permeability (k)	2.4×10^{-5} m/s
Friction angle (ϕ)	33.13°
Apparent cohesion (c)	0.309 kPa
Interface friction angle of humid sand on Aluminum(ϕ_i)	25.4°
Interface adhesion of humid sand on Aluminum (c_i)	0.22 kPa
Interface friction angle of humid sand on Teflon sheet (ϕ_t)	29.0°
Interface adhesion of humid sand on Teflon sheet (c_t)	0.006 kPa

5.2.1 Instrumentation

The test conditions regarding the geometry are elaborated in Table 5-2. For monitoring purposes, seven pore water pressure sensors and nine earth pressure gauges were utilized and positioned according to the configuration illustrated in Figure 5-6. Earth pressure sensors were positioned on the bedding plane and lateral support near the bedding plane. Meanwhile, pore water pressure transducers were placed on the bedding to observe changes in seepage during water supplementation. Additionally, three laser sensors were deployed to track soil movement throughout the experiments. The first laser sensor was directed towards the surcharge to detect surcharge movement during its application. The second laser sensor was aimed at the middle slope to observe slope upheaval during slope collapse. Lastly, the third laser sensor was directed towards toe support to observe toe slope movement during slope collapse. Figure 5-7 illustrates the laser sensors' orientations in each experiment series.

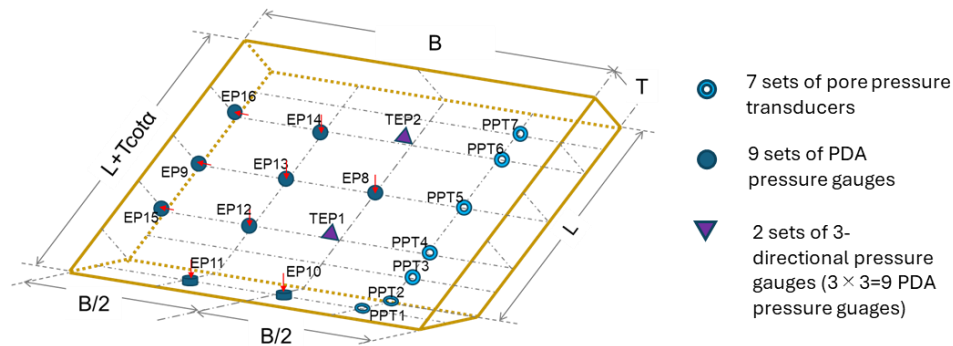
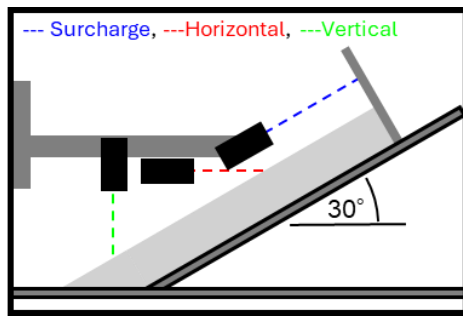
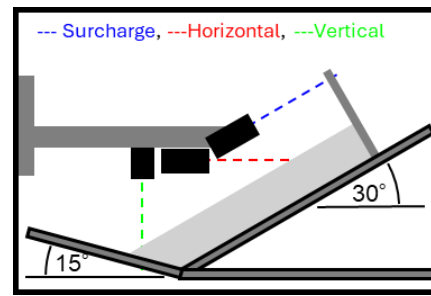


Figure 5-6 The instrumentation set up



(a) slope angle of 30 degrees with a reverse toe angle of 0 degrees



(b) slope angle of 30 degrees with a reverse toe angle of 15 degrees

Figure 5-7 Pointing position of laser sensors

Table 5-2 Configuration of slope model lying bedding plane with lateral, and toe supports.

Test no.	Slope angle α (°)	Toe angle β (°)	Thickness (cm)	Width (cm)	Length (cm)	Surcharge weight(kg)	n_g
1	30	0	3	40	20	4.5	50
2	30	0	3	40	20	4.5	50
3	30	0	3	40	20	4.5	50
4	30	0	5	40	20	6.8	50
5	30	15	3	40	20	4.7	50
6	30	15	3	40	20	4.7	50

5.2.2 Preparation of centrifuge physical model

The preparation of the physical model commenced with adjusting the Hiroshima sand to a water content of 8% and a specific weight. Subsequently, pore water pressure sensors and earth pressure sensors were meticulously installed on the Teflon sheet and aluminum plate, as illustrated in Figure 5-8. Following this, the soil chamber underwent rotation by a crane to orient the bedding plane horizontally, facilitating compaction.

Once the bedding plane was properly aligned, a wooden plate was affixed to constrain the model to the desired length of the slope. The adjusted Hiroshima sand was then carefully placed into the prepared mold, as depicted in Figure 5-9, and compacted within delineated lines to regulate the compaction degree, as shown in Figure 5-10. Upon completion of compaction, the soil chamber was rotated back to its original position, and laser sensors were installed according to Figure 5-6. The positions of three laser sensors are depicted in Figure 5-7, pointing to the surcharge cart, middle slope (horizontal), and toe slope(vertical).

Subsequently, the surcharge load cart and water supplementation apparatus were installed to induce slope failure. A top water tank was then positioned to cover the soil

chamber and securely fastened with nuts. Following these preparations, the soil chamber was weighed and installed into the centrifuge pit for experimentation.

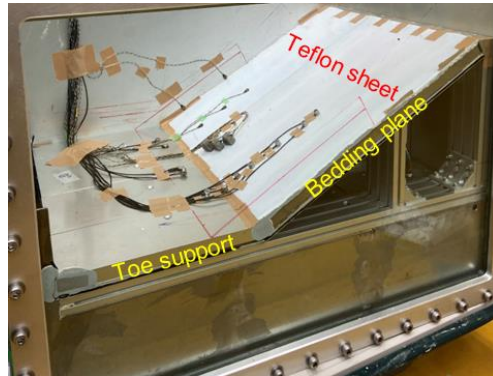


Figure 5-8 Installation of instruments

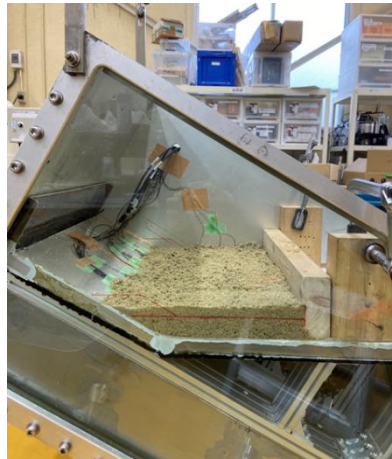


Figure 5-9 Restricted mold and construction of slope model

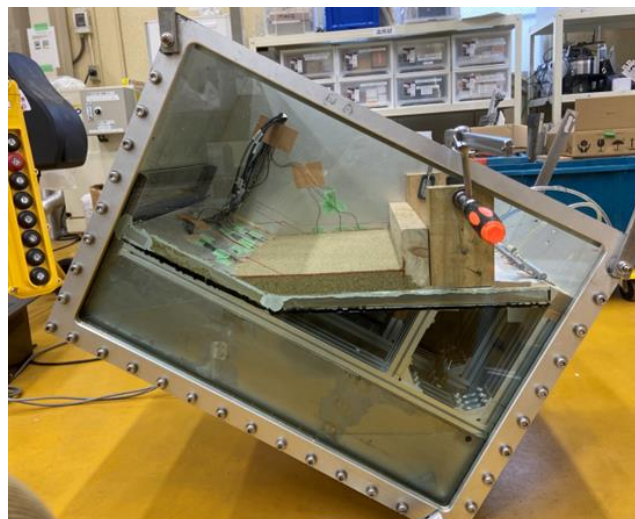


Figure 5-10 A slope after compaction

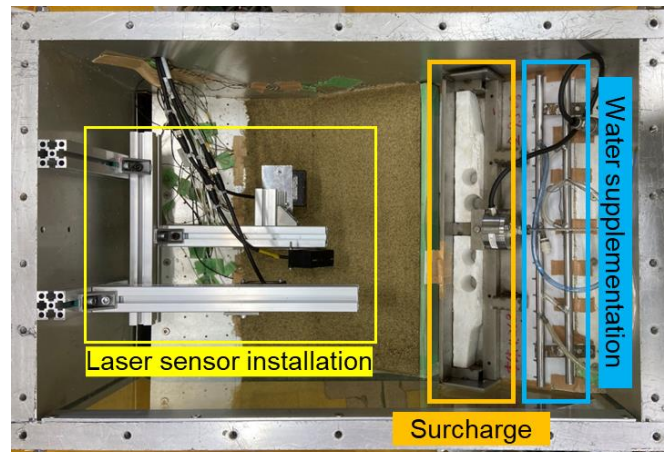


Figure 5-11 Installation of laser sensor, surcharge and water supplementation.



Figure 5-12 Installation of supply top tank

5.3 Method

Experimental procedure is shown in Figure 5-13. Once the soil model was installed in the centrifuge room and connected to the data logger, centrifuge acceleration was increased to the target of 50G and maintained. The slope model was then triggered to fail using a combination of surcharge and water. However, the sequence of adding surcharge and water could vary. If water was added first without triggering slope failure, surcharge was subsequently added, and the load cell recorded the applied force on the slope model during surcharge release. Conversely, if surcharge was added first and slope failure did not occur, water was then added to induce slope failure. If slope failure occurred after adding surcharge first, the experiment was conducted under dry conditions, with surcharge triggering the failure. During slope failure, high-speed cameras were employed to record the movement of the slope from a lateral perspective.

Six experiments were conducted under the following conditions:

Test 1: 50G, watering, surcharge

Test 2: 50G, watering, surcharge

Test 3: 50G, surcharge

Test 4: 50G, surcharge, watering

Test 5: 50G, surcharge

Test 6: 50G, watering, surcharge

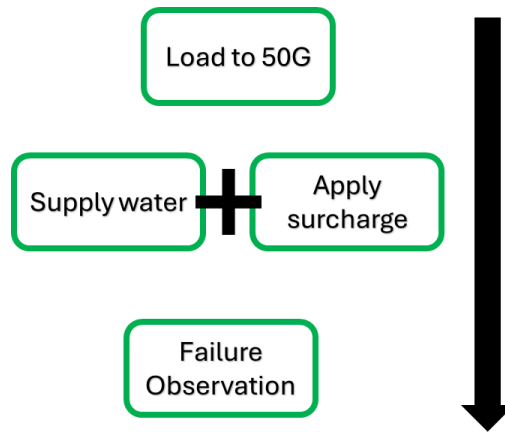


Figure 5-13 Experimental procedure

5.4 Experimental results

After completing the experiment, the data collected from various sensors including laser sensors (measuring displacement in mm), load cell (recording drag force in N), earth pressure sensors (measuring in kPa), and pore water pressure sensors (measuring in kPa) were analyzed. The elapsed time was recorded and displayed only during slope failure. The experimental results are comprehensively elucidated through the detailed analysis of Tests 1 to 6.

5.4.1 Test 1: Geometry condition $\alpha=30^\circ$, $\beta=0^\circ$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, watering, surcharge.

Figure 5-14 presents the experimental results for Test 1. Upon reaching a centrifugal acceleration of 50G and stabilizing, water was supplied to the slope. During this phase, there were no discernible changes in displacement or earth pressure. However, the drag force remained constant as the surcharge 4.5kg was not yet added to the slope and was held by the motor.

Subsequently, upon adding the surcharge, there was a significant increase in vertical displacement observed at the toe slope, indicating toe sliding occurred. Notably, there were no discernible signals of surcharge and horizontal displacement during the experiment. The increase in drag force resulted from the surcharge making contact with

the slope and exerting a force, different from the drag force observed before and after the surcharge addition, allowing for the calculation of the applied force.

Furthermore, earth pressure notably increased at EP12, 13, and 14, indicating slope sliding. However, the earth pressure at other points remained consistent. Pore water pressure increased during the water supplementation phase but decreased upon applying the surcharge. Thus, the data analysis focused on the time of failure to calculate the slope stability using theoretical calculations.

Figure 5-15 illustrates the slope failure following the completion of the centrifuge experiment. The movement predominantly occurred at the toe of the slope, indicating that there was no significant thrust or upheaval at the crest of the slope. This observation supports the displacement results shown in Figure 5-14(a). Additionally, during the loading of the surcharge cart onto the slope, the movement of surcharge cart was symmetrical, as depicted in Figure 5-15(a).

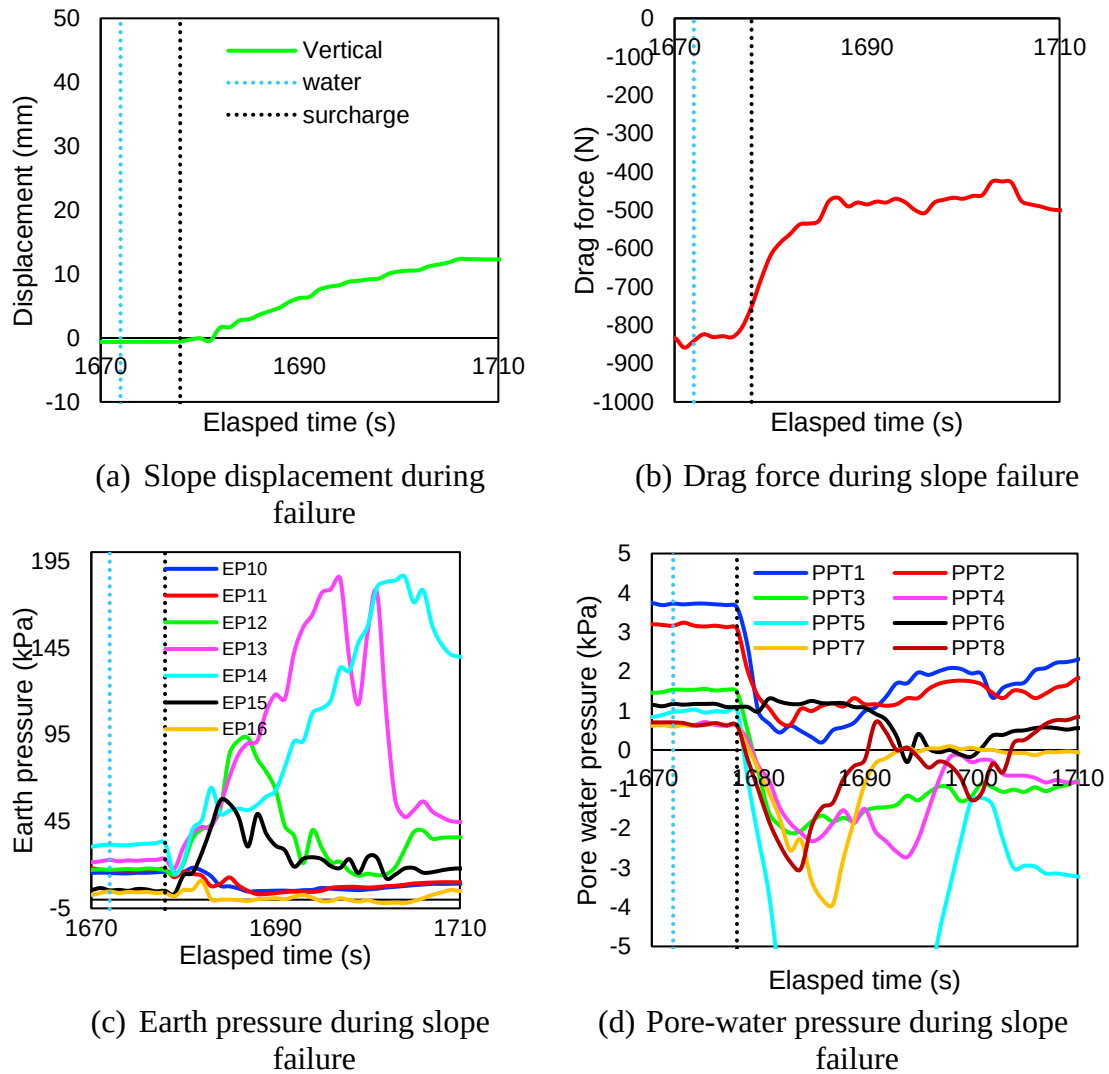
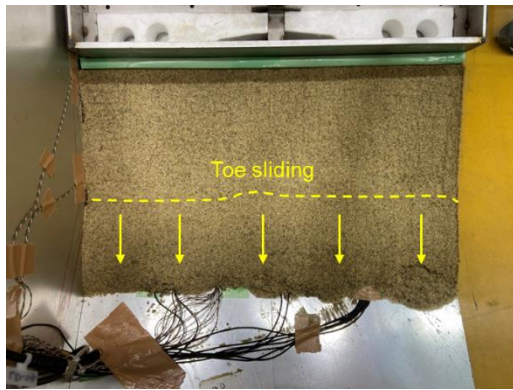


Figure 5-14 Experimental result of test 1



(a) Top view of slope.



(b) Side view of slope

Figure 5-15 Slope failure observed after the testing of Test 1.

5.4.2 Test 2: Geometry condition $\alpha=30^\circ$, $\beta=0^\circ$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, watering, surcharge.

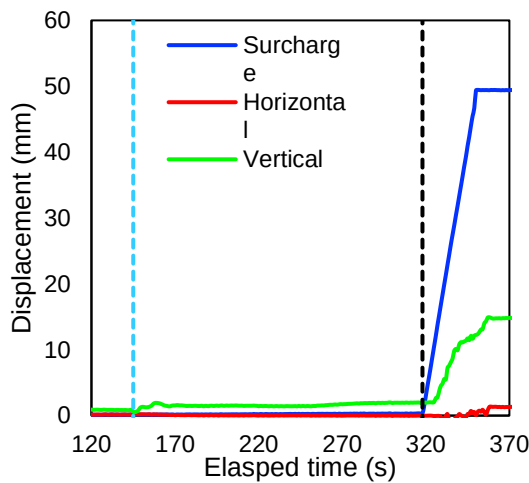
Figure 5-16 depicts the experimental results for Test 2. Following the attainment of a centrifugal acceleration of 50G and stabilization, water was introduced to the slope. Throughout this phase, there were no observable alterations in displacement or earth pressure. However, the drag force remained constant since the surcharge of 4.5kg had not yet been added to the slope and was held in place by the motor.

Subsequently, upon the addition of the surcharge, there was a notable elevation in the laser sensor reading corresponding to the surcharge position. Furthermore, a significant increase in vertical displacement was observed at the toe slope, indicative of toe sliding. However, minimal movement was detected in the horizontal laser sensor located at the middle slope, suggesting the absence of thrust or upheaval. Consequently, based on the laser sensor data, it can be confirmed that toe sliding exclusively contributed to the slope failure.

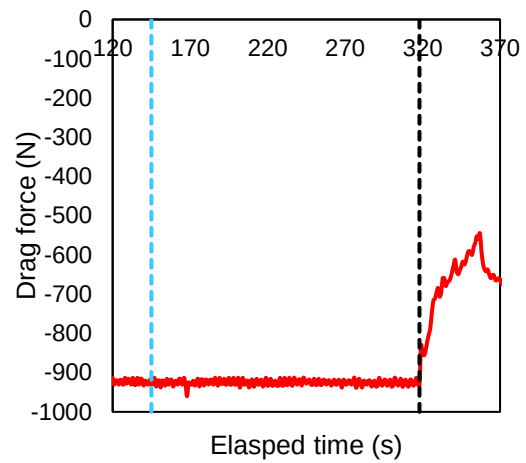
The rise in drag force was attributed to the surcharge contacting the slope and exerting a force. The disparity between the drag force recorded before (-938N) and after (-706.6N) the addition of the surcharge facilitated the calculation of the applied force (231.4N).

Moreover, earth pressure notably surged at EP12, 13, and 14, indicating slope sliding. Conversely, EP10 and 11 experienced a slight decrease following the addition of the surcharge. Pore water pressure increased during the water supplementation phase but decreased upon the application of the surcharge. Consequently, the analysis focused on the time of failure to determine slope stability using theoretical calculations.

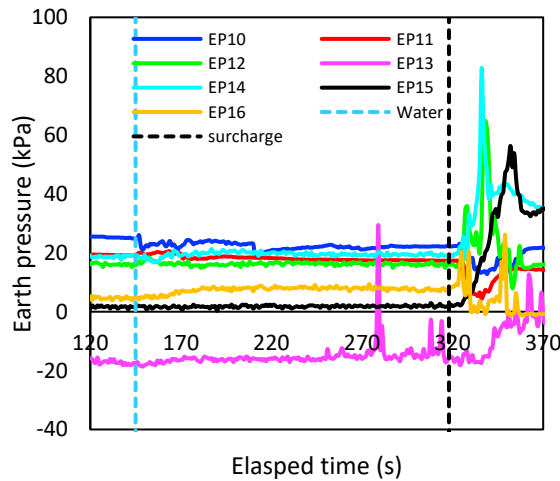
Figure 5-17 depicts the slope failure observed after the conclusion of the centrifuge experiment. The movement primarily occurred at the toe of the slope, suggesting the absence of significant thrust or upheaval at the crest of the slope. This observation aligns with the displacement findings presented in Figure 5-16(a). Furthermore, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was symmetrical, as illustrated in Figure 5-17(a).



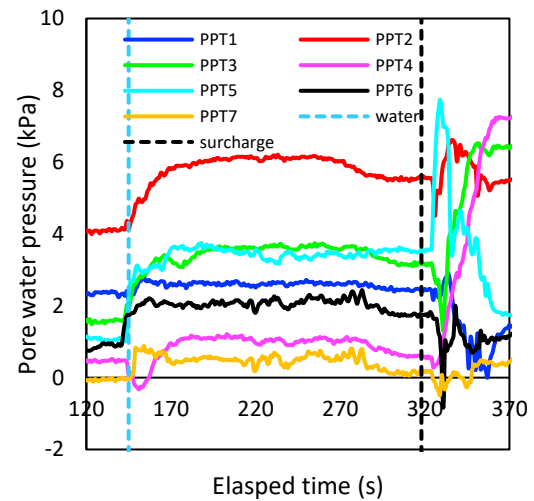
(a) Slope displacement during failure



(b) Drag force during slope failure

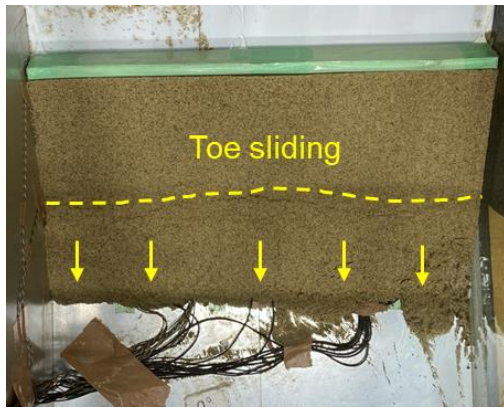


(c) Earth pressure during slope failure



(d) Pore water pressure during slope failure

Figure 5-16 Experimental result of test 2



(a) Top view of slope.



(b) Side view of slope

Figure 5-17 Slope failure observed after the testing of Test 2.

5.4.3 Test 3: Geometry condition $\alpha=30^\circ$, $\beta=0^\circ$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, surcharge.

Figure 5-18 illustrates the experimental results for Test 3, which differed from Tests 1 and 2 in that surcharge alone was used to trigger soil failure, without the addition of water (dry condition). As the centrifugal acceleration reached 50G, the surcharge 4.5kg was held by a motor, and the load cell recorded a drag force of -584.7N.

Upon adding the surcharge, there was a noticeable increase in the laser sensor readings corresponding to the position of the surcharge. Moreover, significant increases were observed in both vertical (toe slope) and horizontal (middle slope) displacements, indicating toe sliding and thrust failure.

The increase in drag force was attributed to the surcharge making contact with the slope and exerting a force. The difference in drag force recorded before (-584.7N) and after (-112N) the addition of the surcharge allowed for the calculation of the applied force (471.7N).

During the addition of the surcharge to the slope, earth pressure increased on EP15 and EP16 but decreased on EP10 to EP14. Since water was not supplied in this experiment, pore water pressure was disregarded for calculation. However, pore water pressure decreased during the addition of the surcharge due to the decrease in earth pressure resulting from slope sliding. Consequently, the analysis focused on the time of failure to determine slope stability using theoretical calculations.

Figure 5-19 depicts the slope failure observed after the conclusion of the centrifuge experiment. The movement occurred both at the toe and the crest of the slope, indicating simultaneous toe sliding and thrust failure. This observation aligns with the vertical and horizontal displacement findings presented in Figure 5-18(a). Furthermore,

during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was not symmetrical, as illustrated in Fig. 5-19(a).

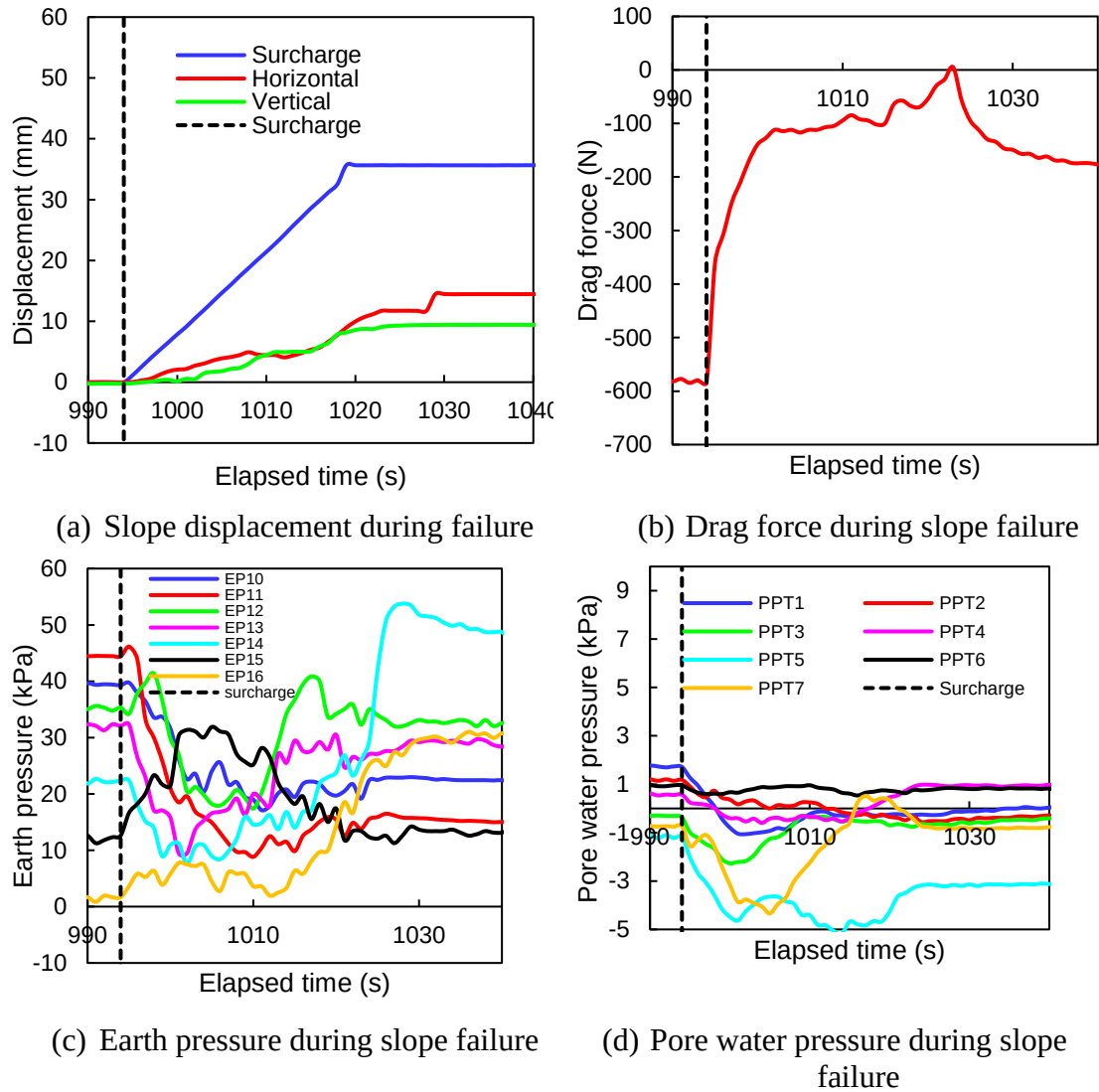


Figure 5-18 Experimental result of test 3

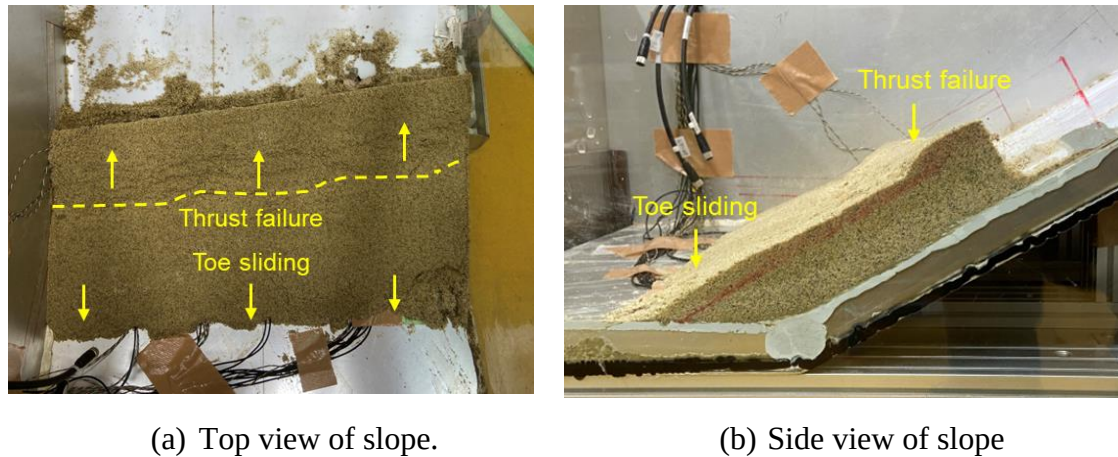


Figure 5-19 Slope failure observed after the testing of Test 3.

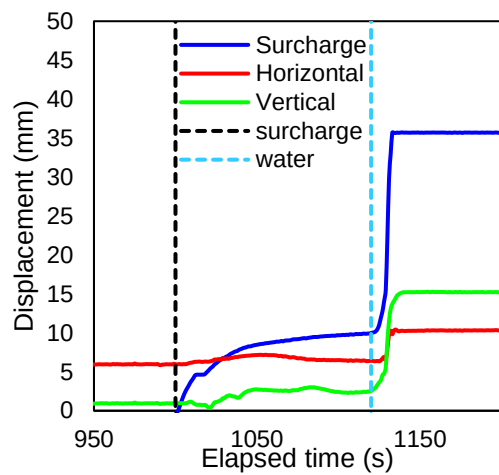
5.4.4 Test 4: Geometry condition $\alpha=30^\circ$, $\beta=0^\circ$, $T = 5\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, surcharge, watering.

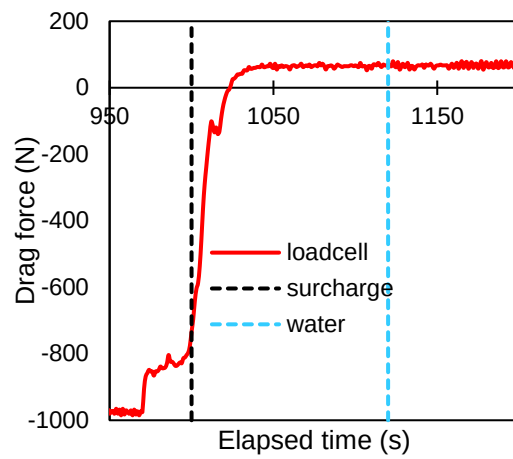
Test 4 featured a distinct geometry due to a thickness of 5 cm, resulting in an increased surcharge of 6.8 kg. Following the established procedure, the surcharge was applied to the slope before water supplementation, as the surcharge alone was insufficient to induce slope failure. Figure 5-20 presents the experimental outcomes for Test 4. Upon reaching a centrifugal acceleration of 50G and stabilizing, the surcharge was introduced to the slope. The drag force reached zero, indicating complete application of the surcharge to the slope. The difference between the drag force readings before (-975.3N) and after (+64.7N) the surcharge addition enabled the calculation of the applied force (1040N). Subsequently, a notable increase in the laser sensor reading corresponding to the surcharge position was observed. Minimal movement was detected in both vertical and horizontal laser sensors, suggesting the absence of failure. However, changes in earth pressure and pore water pressure were noted.

Following the introduction of water to the slope, a significant increase in vertical displacement (toe slope) occurred, with minimal horizontal movement. Large vertical and small horizontal displacements indicated toe sliding. Additionally, pore water pressure increased, suggesting parallel seepage. While earth pressure increased at EP12, 13, 14, and 15, it minimally decreased at EP10, 11, and 16. The analysis thus focused on failure time to determine slope stability using theoretical calculations.

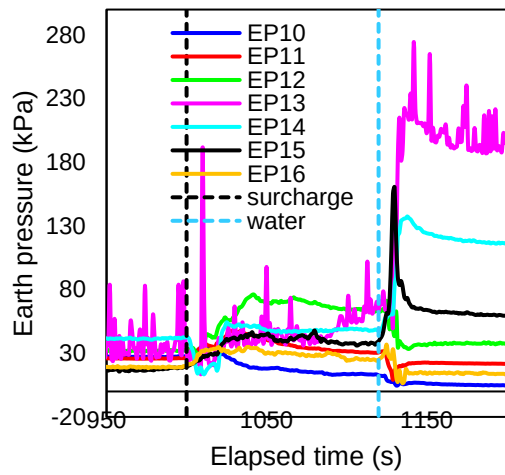
Figure 5-21 illustrates the slope failure observed post-experiment. Predominant movement occurred at the toe of the slope, indicating minimal thrust or upheaval at the crest. This observation aligns with the displacement findings in Figure 5-20(a). Furthermore, during the loading of the surcharge cart onto the slope, symmetrical movement of the surcharge cart was observed, as depicted in Figure 5-21(a).



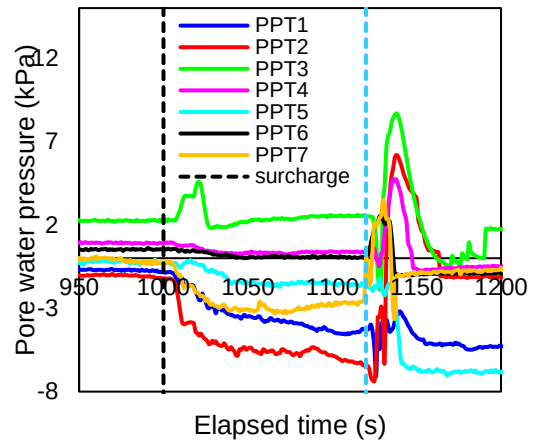
(a) Slope displacement during failure



(b) Drag force during slope failure

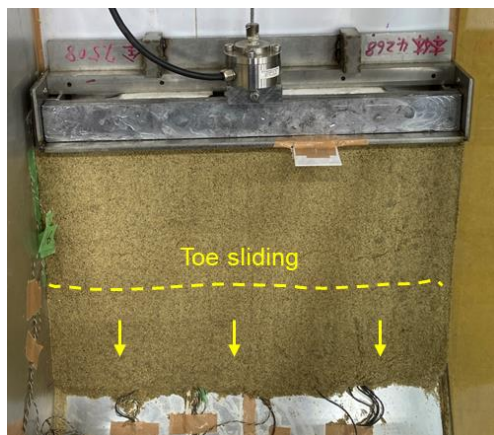


(c) Earth pressure during slope failure

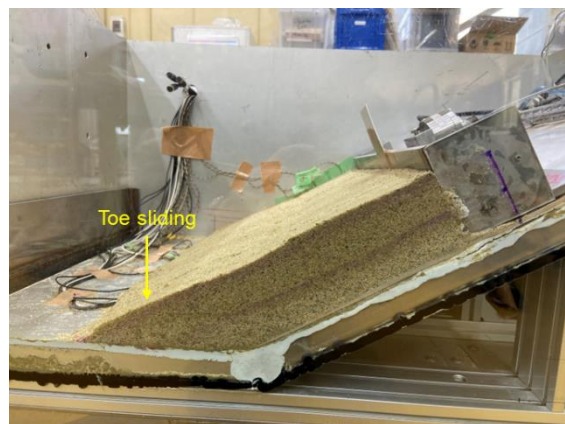


(d) Pore water pressure during slope failure

Figure 5-20 Experimental result of test 4



(a) Top view of slope.



(b) Side view of slope

Figure 5-21 Slope failure observed after the testing of Test 4.

5.4.5 Test 5: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, surcharge.

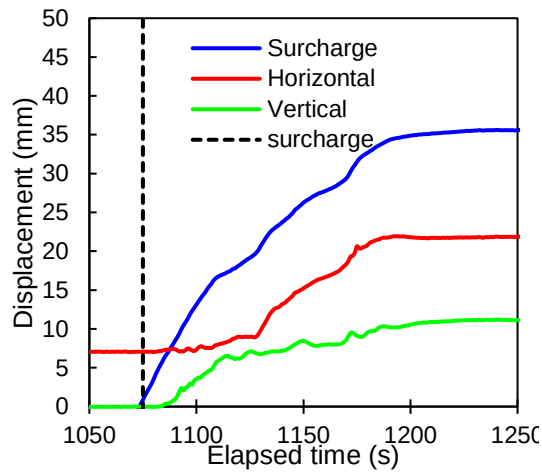
Test 5 involved a 15-degree reverse toe angle, with the surcharge solely triggering slope failure. The experimental outcomes are depicted in Figure 5-22. Upon reaching a centrifugal acceleration of 50G, a 4.5 kg surcharge was held by a motor, and the load cell recorded a drag force of -395.6N.

Following the introduction of the surcharge, noticeable increases were observed in the laser sensor readings corresponding to the surcharge position. Moreover, significant increases were detected in both vertical (toe slope) and horizontal (middle slope) displacements, indicating toe sliding and thrust failure. The displacement during slope collapse is illustrated in Figure 5-22(a).

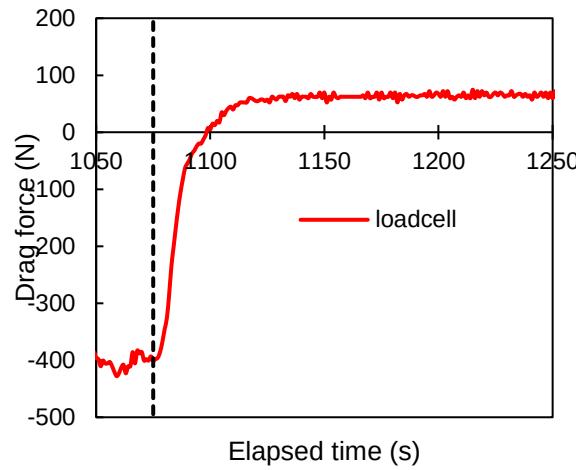
Upon application of the surcharge to the slope, the increase in drag force was attributed to the surcharge contacting the slope and exerting a force. The difference in drag force recorded before (-395.6N) and after (+52.2N) the surcharge addition facilitated the calculation of the applied force (447.8N).

During the surcharge addition, earth pressure increased at EP10, EP11, EP15, and EP16, but decreased at EP12, EP13, and EP14, as. Since water was not supplied in this experiment, pore water pressure was disregarded for calculation. However, pore water pressure decreased during the surcharge addition due to the decrease in earth pressure resulting from slope sliding. Consequently, the analysis focused on the time of failure to determine slope stability using theoretical calculations.

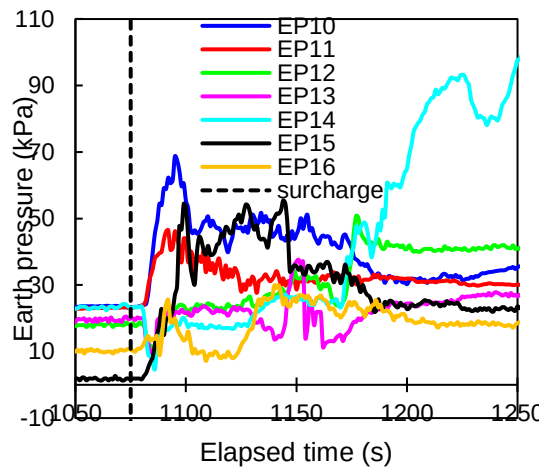
Figure 5-23 depicts the slope failure observed after the conclusion of the centrifuge experiment. The movement occurred both at the toe and the crest of the slope, indicating simultaneous toe sliding and thrust failure. This observation aligns with the vertical and horizontal displacement findings presented in Figure 5-22(a). Furthermore, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was not symmetrical, as illustrated in Figure 5-23(a).



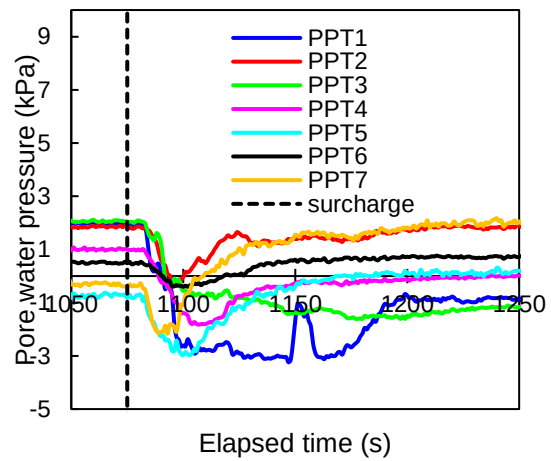
(a) Slope displacement during failure



(b) Drag force during slope failure

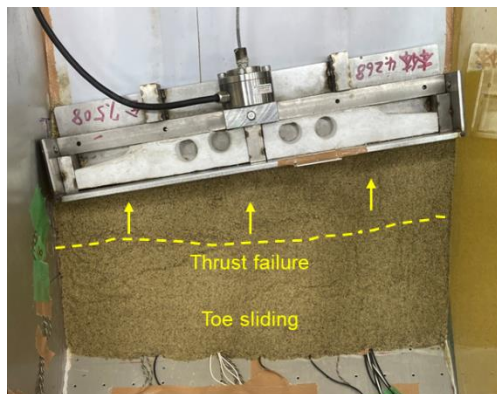


(c) Earth pressure during slope failure



(d) Pore water pressure during slope failure

Figure 5-22 Experimental result of test 5



(a) Top view of slope.



(b) Side view of slope

Figure 5-23 Slope failure observed after the testing of Test 5.

5.4.6 Test 6: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, watering, surcharge.

Test 6 involved a 15-degree reverse toe angle as same as test 5. However, water was supplied to slope first, then followed by surcharge adding. The experimental outcomes are depicted in Figure 5-24.

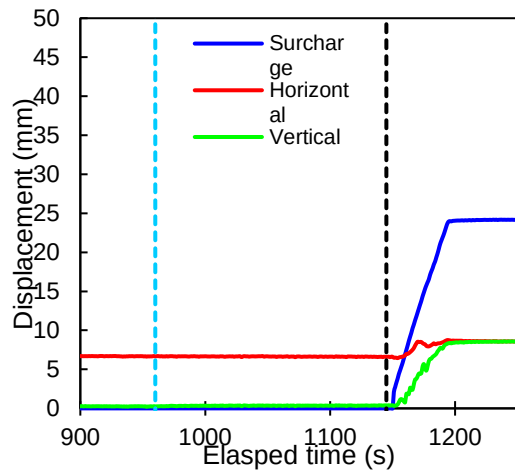
During water supplementation, there were no observable alterations in displacement or earth pressure. However, the drag force remained constant since the surcharge 4.5kg had not yet been added to the slope and was held in place by the motor and record drag force as -427.9N.

Subsequently, upon the addition of the surcharge, there was a notable elevation in the laser sensor reading corresponding to the surcharge position. Furthermore, a significant increase in vertical displacement was observed at the toe slope, indicative of toe sliding. However, minimal movement was detected in the horizontal laser sensor located at the middle slope, suggesting the absence of thrust or upheaval. Consequently, based on the laser sensor data, it can be confirmed that toe sliding exclusively contributed to the slope failure.

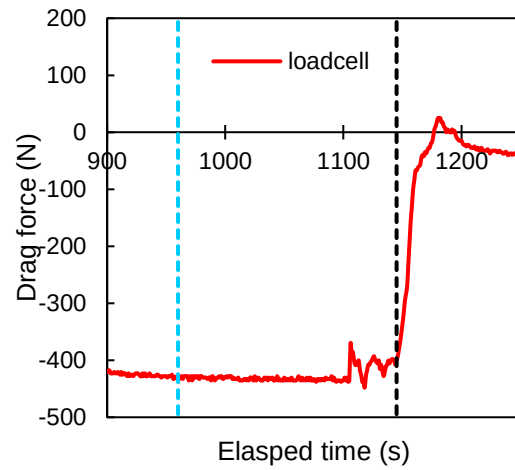
The rise in drag force was attributed to the surcharge contacting the slope and exerting a force. The disparity between the drag force recorded before (-427.9N) and after (-67.2N) the addition of the surcharge facilitated the calculation of the applied force (360.7N).

Moreover, all earth pressure was increased during surcharge adding as shown in Figure 5-24(c). Pore water pressure increased during the water supplementation phase but decreased upon the application of the surcharge, as illustrated in Figure 5-24(d). Consequently, the analysis focused on the time of failure to determine slope stability using theoretical calculations.

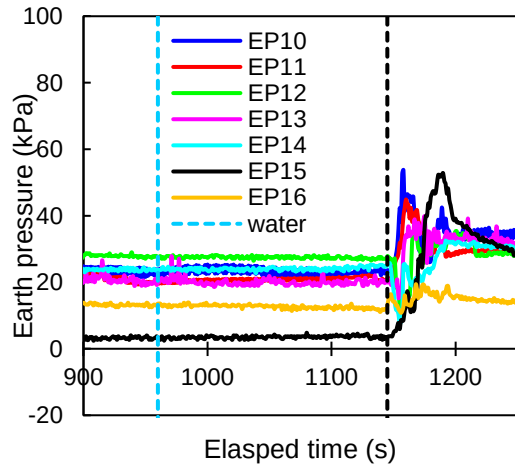
Figure 5-25 illustrates the slope's failure observed post the centrifuge experiment's conclusion. Movement was observed both at the toe and crest of the slope. However, thrust failure on the crest slope occurred subsequent to toe sliding. Thus, the slope failure primarily involved toe sliding during the experimental procedure, as confirmed by the displacement recorded by laser sensors (Figure 5-24(a)). Additionally, during the surcharge cart's loading onto the slope, its movement was asymmetrical, as depicted in Figure 5-25(a).



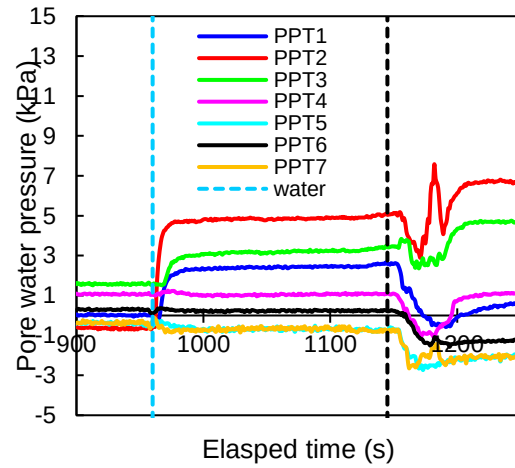
(a) Slope displacement during failure



(b) Drag force during slope failure

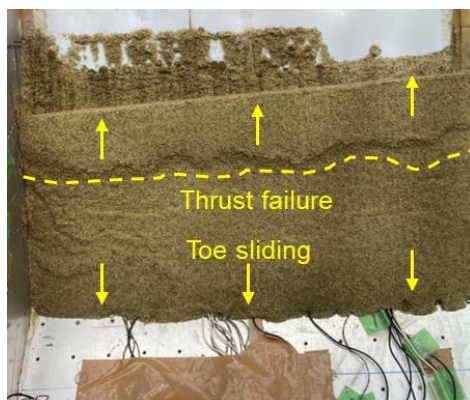


(c) Earth pressure during slope failure

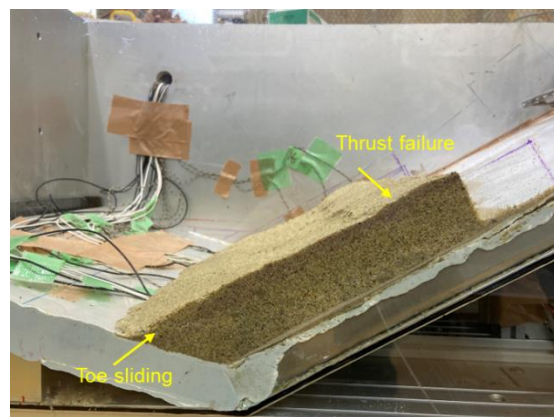


(d) Pore water pressure during slope failure

Figure 5-24 Experimental result of test 6



(a) Top view of slope.



(b) Side view of slope

Figure 5-25 Slope failure observed after the testing of Test 6.

5.4.7 Experimental summation

The experimental results include the applied force recorded by the load cell and the pore water pressure under a centrifugal acceleration of 50. During the collapse of the slope model, the applied force and pore water pressure were collected and summarized in Table 5-3. Pore-water pressure was analyzed separately for the toe and middle slopes to align with the theoretical framework depicted in Figure 5-4. The toe pore water pressure readings were obtained from PPT2, while the pore water pressure for the middle slope was averaged from PPT4 to PPT7. The applied force was determined by calculating the difference before and after loading.

Furthermore, Table 5-3 presents the geometric conditions and failure modes. For slopes lying on a bedding plane with lateral and toe support, two failure modes were observed: toe sliding and thrust & toe failure. Thrust failure was clearly evident in dry conditions where water supplementation was not applied. In contrast, toe sliding occurred in tests involving watering, indicating that groundwater seepage affects the failure mode of slopes lying on a bedding plane. Additionally, the test results will be utilized to calculate the slope stability and subsequently compared with theoretical predictions.

Table 5-3 The experimental results of dip slope under the influence of groundwater seepage.

Test no.	Slope angle α (°)	Toe angle β (°)	Thickness (cm)	Surcharge weight(kg)	Applied force(N)	n_g	Pore water pressure at toe, U_2 (kPa)	Pore water pressure at slope, U_1 (kPa)	Failure mode
1	30	0	3	4.5	315.976	50	3.71	1.13	Toe sliding
2	30	0	3	4.5	263.728	50	6.62	2.09	Toe sliding
3	30	0	3	4.5	584.7	50	0	0	Thrust & Toe
4	30	0	5	6.8	1055	50	1.88	0.42	Toe sliding
5	30	15	3	4.7	457.8	50	0	0	Thrust & Toe
6	30	15	3	4.7	425.4	50	5.04	0.68	Tor sliding

5.5 PIV analysis

During the experiment's procedures, the slope collapsed rapidly, necessitating the deployment of a high-speed camera to document the event. This camera, operating at 100 frames per second and with a resolution of 1244x890 pixels, offered exceptional image quality. It captured images of the slope profile through the transparent side wall of the

model chamber, focusing on the specific behavior of the slope model. This targeted approach ensured precise recording of critical details during the failure process. Consequently, the camera captured the slope's movement with remarkable clarity and fluidity, offering valuable insights into the dynamics of slope failure. The removal of the light reflection plate due to the significant size of the soil chamber led to some light being reflected onto the crest slope, as shown in Figure 5-26(a). Consequently, this reflection affected the PIV analysis, resulting in its absence, as illustrated in Figure 5-26(b) and 5-26(c). PIV analysis was conducted on the recorded video footage capturing the slope collapse during each of the tests 1 to 6, as demonstrated in Figure 5-26 to 5-31.

Test 1: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, watering, surcharge.

Figure 5-26(a) displays an image captured during the slope collapse, with the initial slope geometry lined up for reference. PIV analysis was then conducted to generate velocity vectors, as shown in Figure 5-26(b). These velocity vectors are visible across the entire slope, indicating toe sliding and corroborating the findings of the displacement laser sensors. Furthermore, a color plot of the velocity magnitude was utilized to clearly visualize the results from the PIV analysis, as shown in Figure 5-26(c).

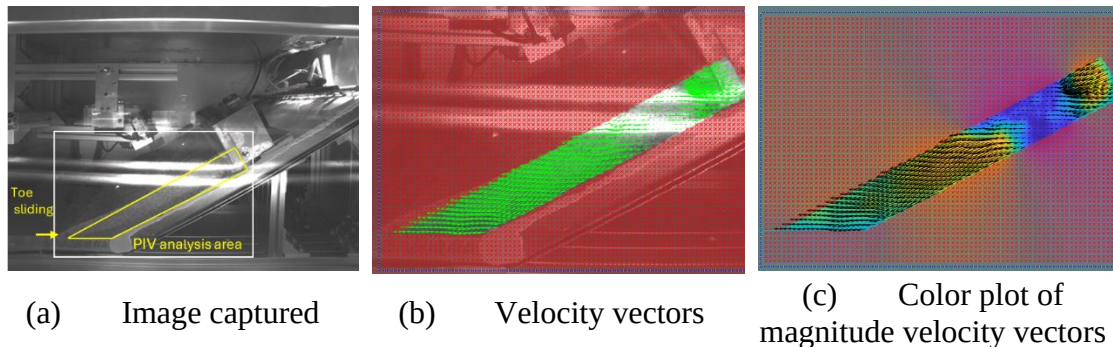


Figure 5-26 PIV analysis results of test 1

Test 2: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, watering, surcharge.

The PIV analysis results for Test 2 mirror those of Test 1 due to the identical experimental conditions. The analysis reveals velocity vectors present across the entire slope, indicating toe sliding. This consistency between the experiments serves to validate the findings of Test 1.

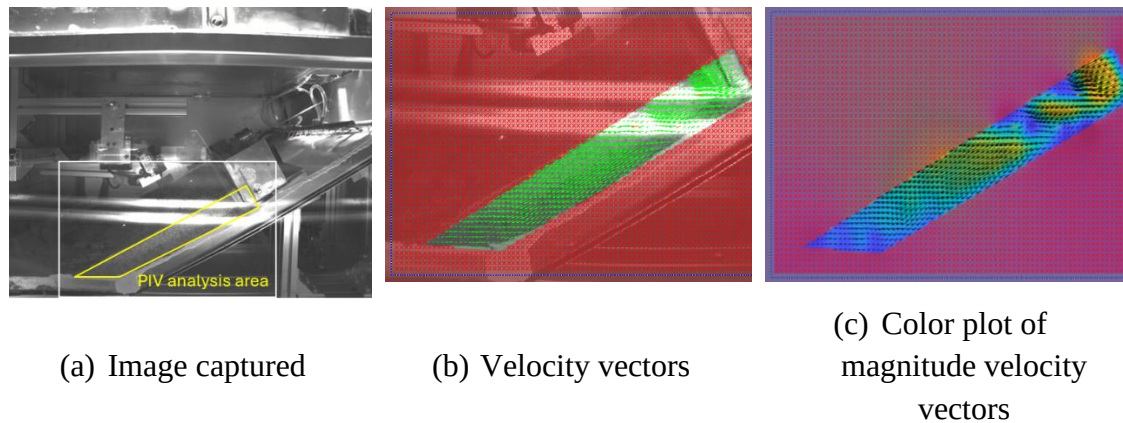


Figure 5-27 PIV analysis results of test 2

Test 3: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, surcharge.

In this experiment, only surcharge loading was employed to trigger slope failure, and watering was not utilized, creating a dry condition. The PIV analysis results revealed velocity vectors appearing from the middle to the crest of the slope, indicating thrust failure during Test 3. The alignment between the PIV results and the displacement data from the laser sensors is notable.

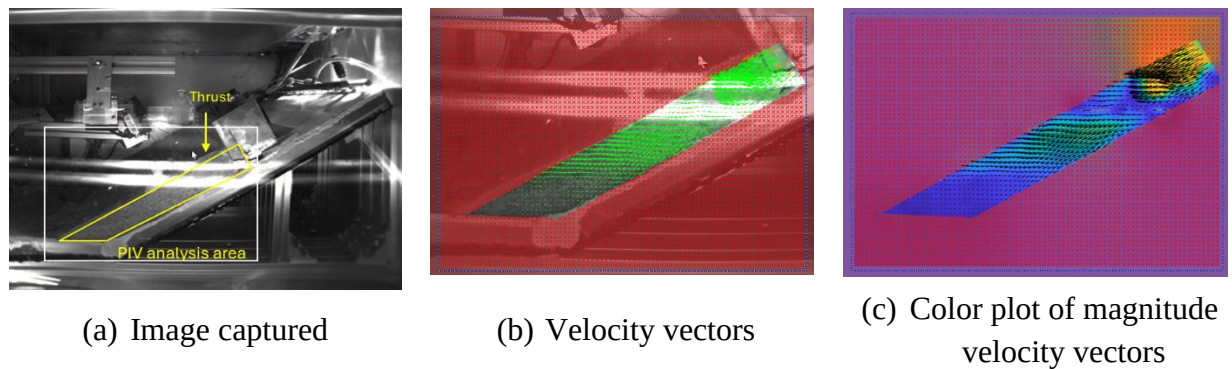


Figure 5-28 PIV analysis results of test 3

Test 4: Geometry condition $\alpha=30$, $\beta=0$, $T = 5\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, surcharge, watering.

This experiment was conducted under geometry conditions with a thickness of 5 cm, resulting in a larger surcharge weight compared to Tests 1 to 3. However, the PIV analysis results showed movement across the entire slope, indicating toe sliding. There was good agreement between the displacement data from the laser sensors and the velocity vectors obtained from the PIV analysis.

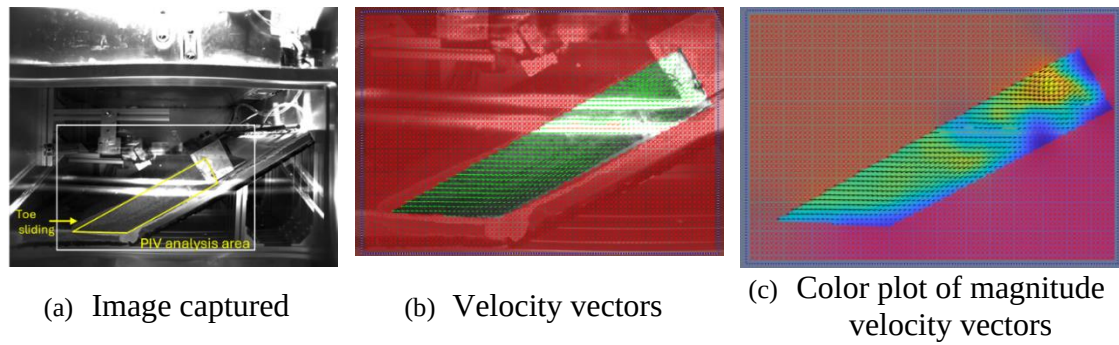


Figure 5-29 PIV analysis results of test 4

Test 5: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, surcharge.

In Test 5, a reverse toe angle of 15 degrees was employed. The test procedure involved solely the use of surcharge to trigger the slope model, indicating dry conditions. Figure 5-30(b) and 5-31(c) depict velocity vectors appearing from the middle slope to the crest of the slope, indicating thrust failure. The results of the PIV analysis align well with the displacement data from the laser sensors.

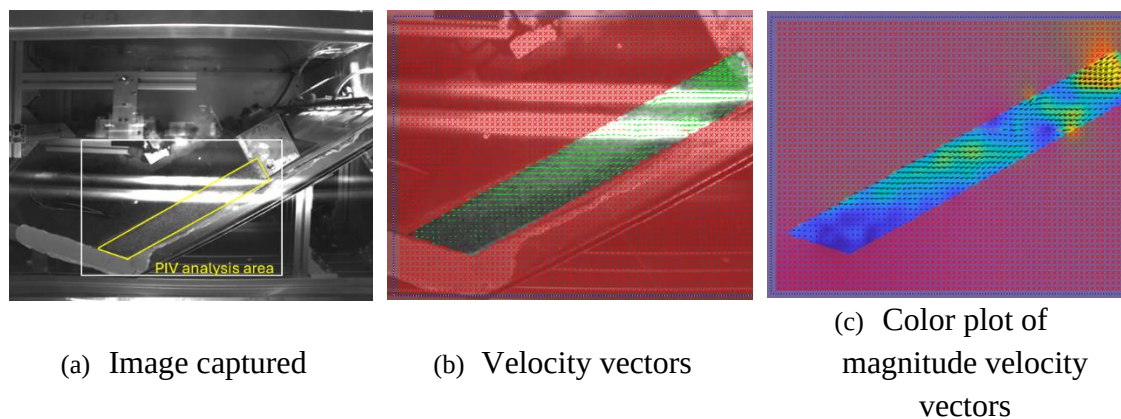


Figure 5-30 PIV analysis results of test 5

Test 6: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, watering, surcharge.

In Test 6, a reverse toe angle of 15 degrees was employed, similar to Test 5. However, in this test, water was supplied before loading the surcharge. Consequently, the PIV analysis results show velocity vectors appearing across the entire slope, indicating toe sliding during the experiment. Furthermore, the PIV results closely align with the displacement data from the laser sensors.

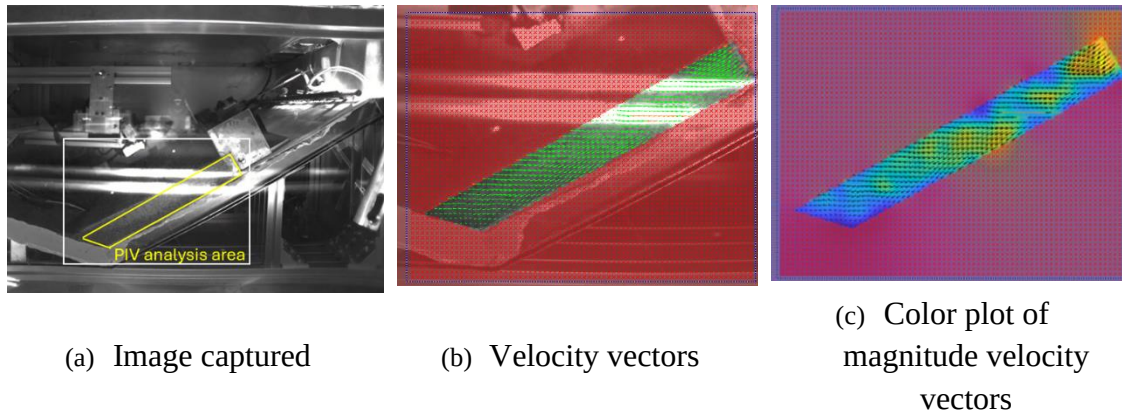


Figure 5-31 PIV analysis results of test 6

5.6 Reduction factor

The reduction ratio (r_d) is calculated using data obtained from earth pressure gauges and pore water pressure sensors installed at specific locations on the slope model. These sensors are strategically positioned to capture the changes in pressure and water content during the experiment. Typically, for calculating the reduction ratio, the earth pressure gauges are placed at various location along the bedding plane (vertical) and lateral support (horizontal), while the pore water pressure sensors are installed at the same layer as earth pressure to indicate effective earth pressure, as expressed Eq. (5-7). These sensors provide crucial data that is used to determine the reduction ratio, which in turn helps in understanding the stability of the slope under different conditions.

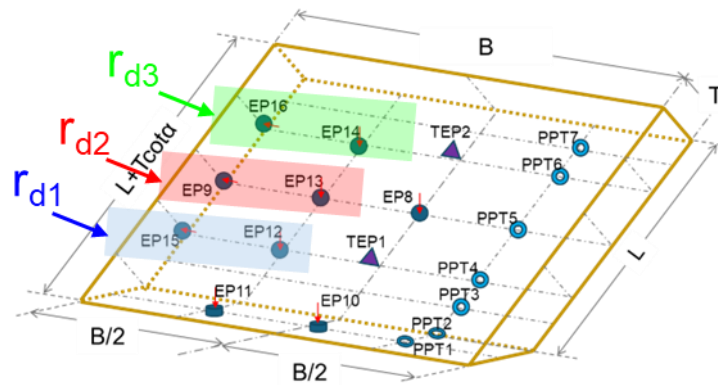


Figure 5-32 location of earth pressure gauge and pore water pressure sensors for calculating reduction ratio

Figure 5-32 illustrates the arrangement of earth pressure sensors and pore water pressure sensors at the same layer, which are utilized to calculate the reduction ratio (r_d). For r_{d1} , EP12 is employed for vertical earth pressure (σ_v), EP15 for horizontal earth

pressure (σ_h), and PPT4 to determine the effective earth pressure (σ_u). For r_{d2} , EP13, EP9, and PPT5 are utilized. Likewise, r_{d3} utilizes EP14, EP16, and PPT6.

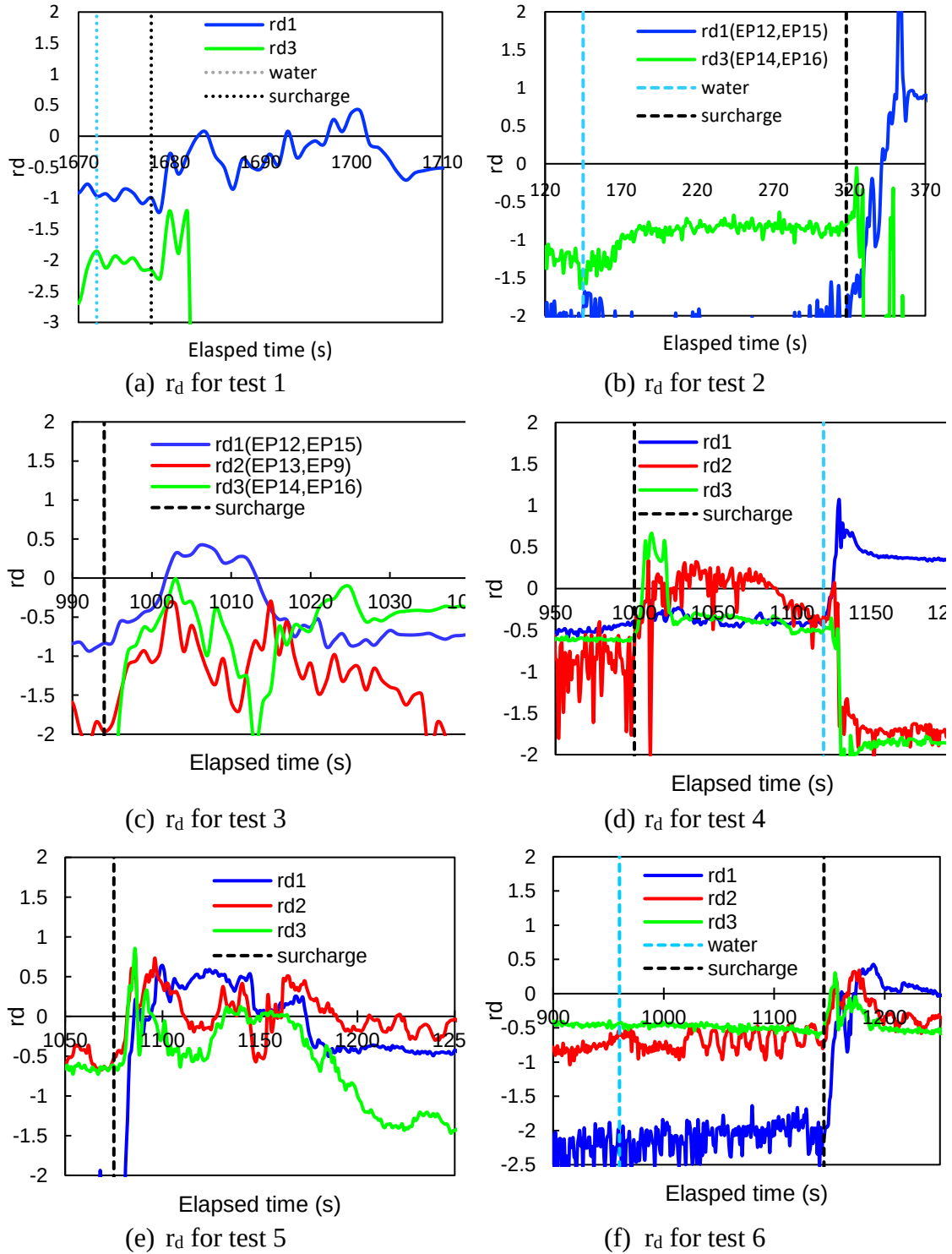


Figure 5-33 Experimental reduction ratio

The experimental reduction ratio(r_d) is depicted in Figure 5-33. In Test 1, shown in Figure 5-33(a), during water supplementation, r_{d1} and r_{d3} remained relatively unchanged. However, both r_{d1} and r_{d3} increased significantly upon the application of the surcharge, reaching a peak at 0.4. This value is crucial for subsequent calculations. In Test 2, during water supplementation, there was minimal change in r_{d1} and r_{d3} . However, upon applying the surcharge to the slope, r_{d1} experienced a substantial increase, peaking at 0.912. Similarly, in Test 3, all r_d values increased immediately upon adding the surcharge, with r_{d1} exhibiting a notable increase and reaching a peak at 0.426. For Test 4, r_{d3} increased during the surcharge addition, but the slope did not fail as indicated by displacement measurements. Upon water supplementation, r_d significantly rose, peaking at 1.06. In Test 5, all r_d values increased immediately after adding the surcharge, peaking at 0.84. Finally, in Test 6, r_d remained unchanged during water supplementation. However, upon applying the surcharge to the slope, all r_d values jumped to 1.

The experimental results indicate an increase in the reduction ratio, peaking around 1, which aligns well with the calculation of passive earth pressure. It is noteworthy that in Tests 1 and 2, r_{d2} could not be calculated due to the absence of a signal from EP9. However, r_{d2} was fixed and displayed consistently in Tests 3 to 6.

5.7 Discussion

In this section, the experimental findings regarding applied force and pore water pressure are summarized in Table 5-3. To validate theoretical calculations, compared with theoretical predictions in Section 5.7.1. The failure mode is discussed in section 5.7.2. Furthermore, a back calculation of the surcharge weight is necessary to demonstrate and validate, as discussed in Section 5.7.3.

5.7.1 Factor of safety (FOS)

The Factor of Safety (FOS) can be calculated based on the resistance force and driving force. During the experiment, pore-water pressure transducers and applied force measurements were recorded. Due to theoretical calculation, slope is divided into slope and toe parts. Therefore, pore-water pressure on toe would be average value of PPT1 and PPT2. For pore-water pressure on slope part and parallel seepage occurred, average from PPT4 to PPT7 are used for calculation. These results can be analyzed in terms of FOS, as illustrated in Figure 5-34. To maximize lateral resistance and predict FOS, a reduction ratio (r_d) of +1 (passive) was assumed. The predicted FOS indicates that slope collapse is likely to correspond with the displacement results obtained from laser sensors. Subsequently, the experimental r_d values were used for calculation and comparison with predictions, as depicted in Figure 5-35.

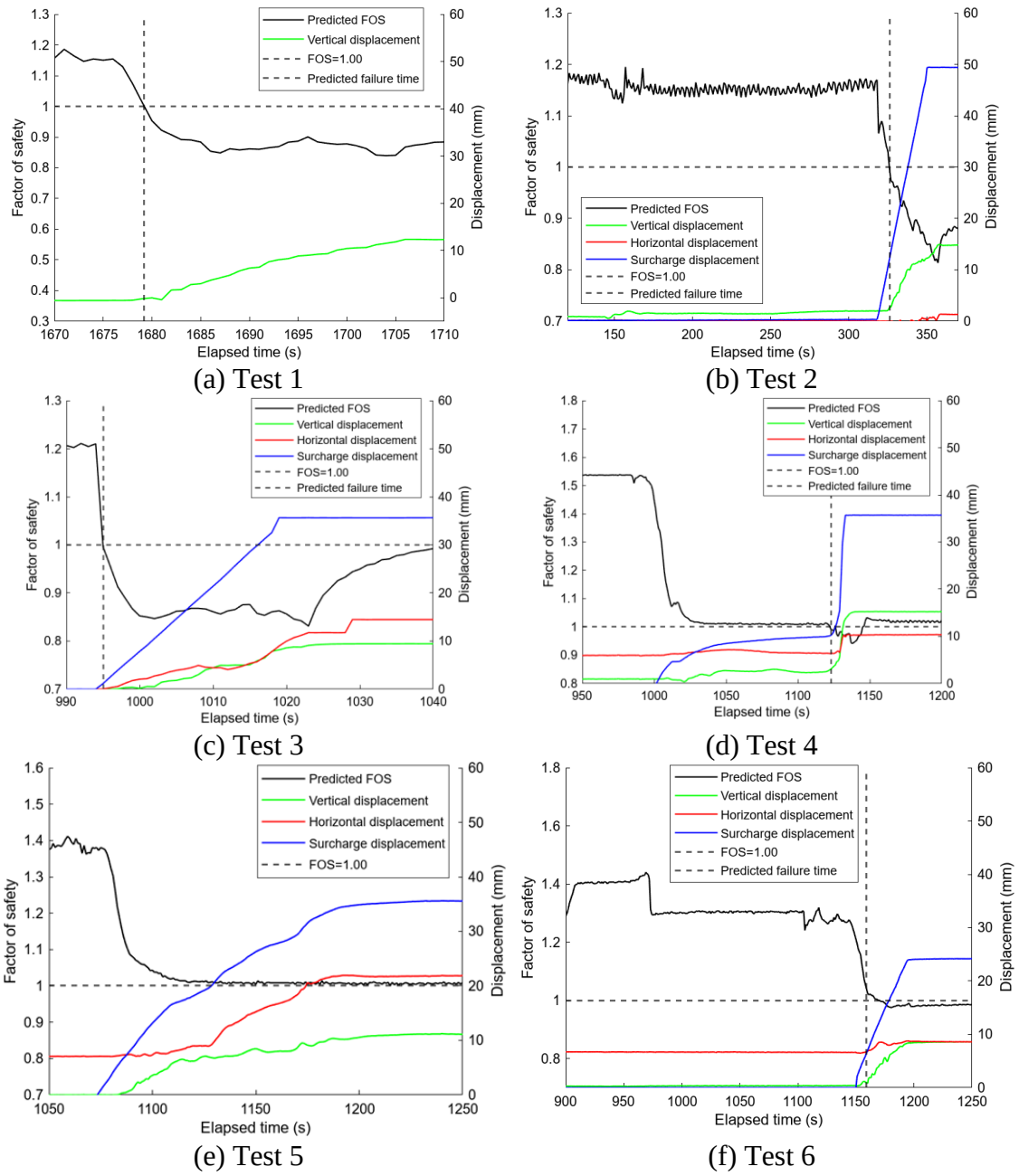


Figure 5-34 Factor of safety subjected pore-water pressure and applied force

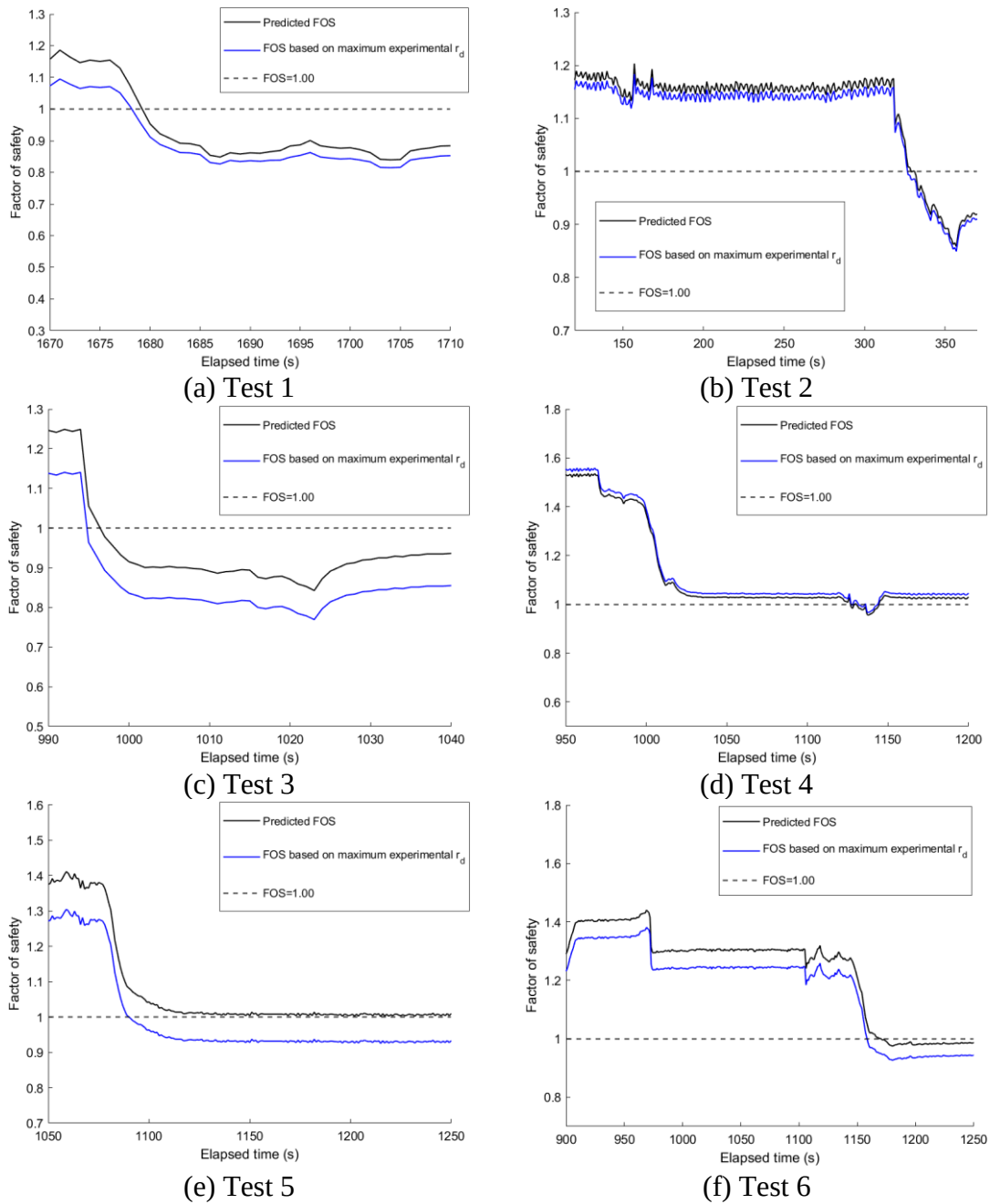


Figure 5-35 Comparison between prediction and experimental factor of safety

5.7.2 Failure mode

In Section 5.1, theoretical calculations are provided for failure mode analysis in Eq. (5-20). Toe resistance (P_s) and passive thrust limitation (P_T) can be used to determine the failure mode of the slope. Figure 5-36 illustrates P_s and P_T ; it can be seen that P_s is lower than P_T , indicating toe sliding. In the experimental results, toe sliding was clearly observed in tests 1, 2, 4, and 6, as shown in Table 5-3. However, tests 3 and 5 exhibited

both toe sliding and thrust failure due to the absence of water application in these tests, highlighting the effect of groundwater.

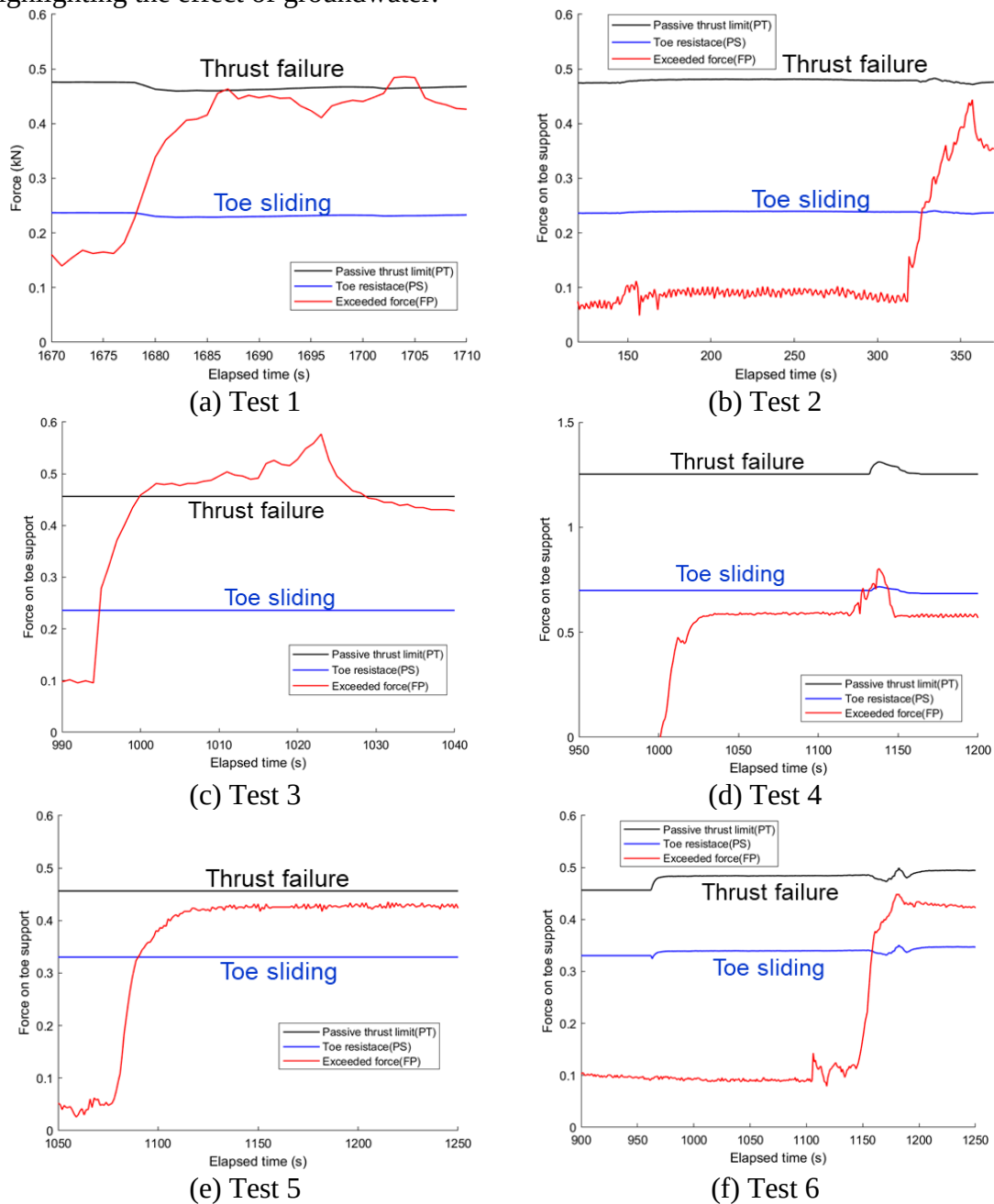


Figure 5-36 Comparison of toe resistance and passive thrust limitation

5.7.3 Back-calculation of surcharge weight

To enhance the accuracy of theoretical calculations, back-calculation of the applied force is of particular interest. Figure 5-37 illustrates the relationship between FOS and applied force. The prediction line represents slope stability considering existing pore-water pressure and $r_d = 1$. The applied force is increased until the FOS reaches 1.00,

indicating slope failure. This predicted applied force is then compared with the applied force that led to slope failure in the experiments.

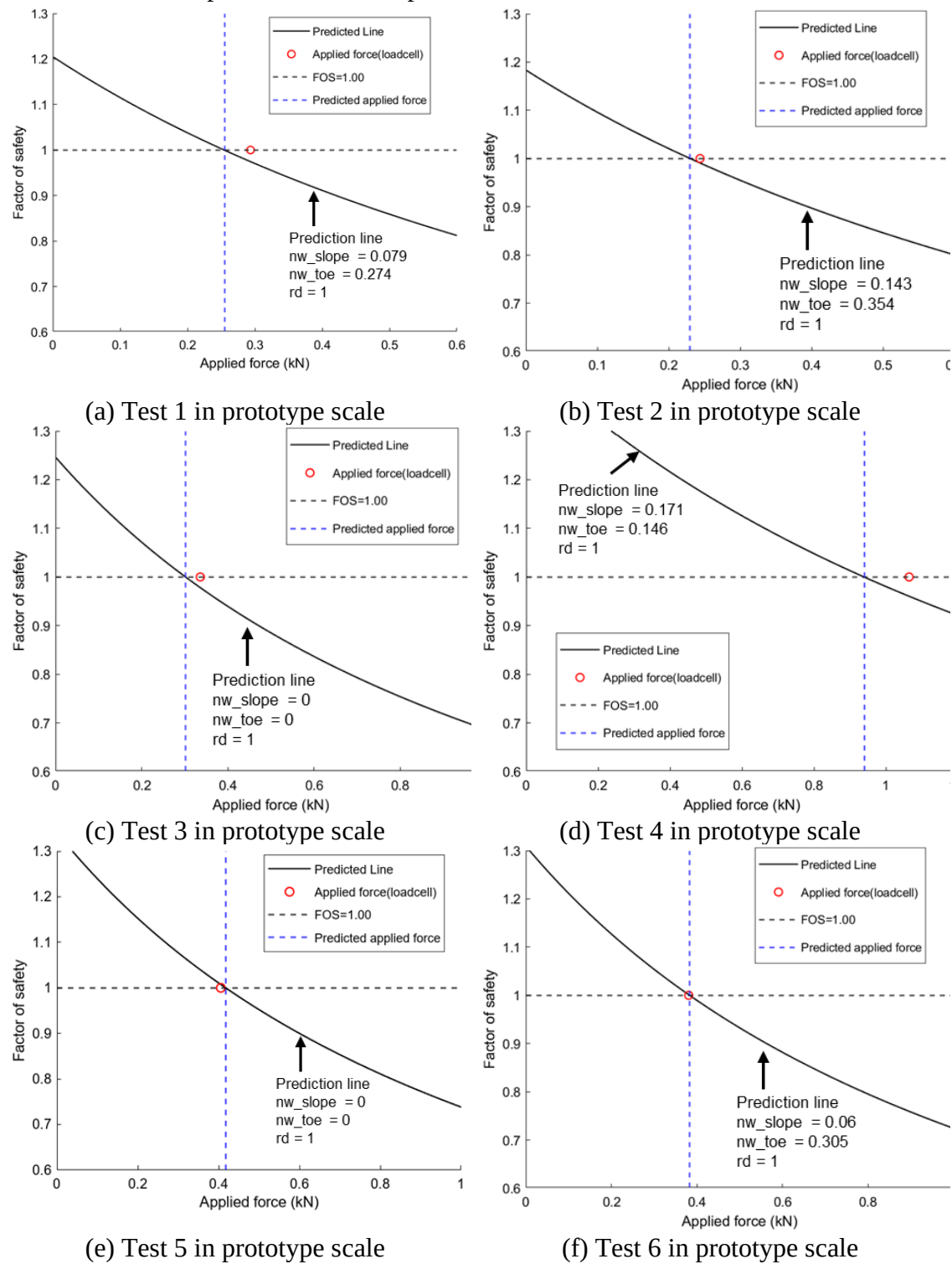


Figure 5-37 Prediction of applied force

The comparison between the predicted applied force and the experimental applied force is shown in Figure 5-38. The coefficient of determination (R^2) is 0.764, indicating that the predictions are reasonably accurate in estimating the applied force and surcharge weight required to trigger failure in the dip slope model under conditions of groundwater seepage.

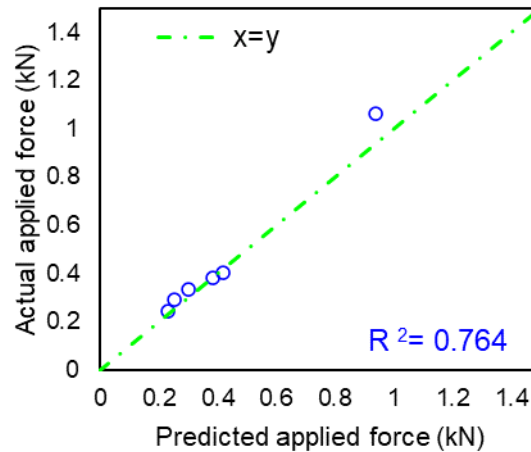


Figure 5-38 Correlation between predicted applied force and actual applied force

5.8 Summary

- Based on the observations made in Chapter 4, it was found that a slope angle of 30 degrees with a reverse toe angle of 20 degrees resulted in fore-thrust failure. However, in Chapter 5, when examining a centrifuge slope model with a slope angle of 30 degrees and a reverse toe angle of 15 degrees under dry conditions, both toe sliding and fore-thrust failure were observed. Conversely, when the same slope model with a reverse toe angle of 15 degrees was subjected to groundwater seepage, toe sliding occurred. These results affirm that groundwater infiltration has a significant effect on altering the mode of failure.
- In Chapter 5, an assessment of the Factor of Safety based on the calculation of resistance force and driving force was conducted. Additionally, parallel seepage was considered to account for failure conditions. The results of the Factor of Safety analysis indicated slope instability leading to slope failure, as corroborated by the displacement data recorded by laser sensors during the experimental procedures.
- Furthermore, an enhancement of the Factor of Safety equation was undertaken to predict the applied force necessary to induce slope failure, utilizing existing pore-water pressure data.

Chapter 6 Centrifugal Model for Counterweight Mitigating Dip Slope with Groundwater Seepage

In this chapter, a series of centrifuge tests were conducted to study counterweight mitigation of dip slopes subjected to groundwater seepage on an impermeable bedding plane. Based on the experimental results from Chapter 5, toe sliding predominantly occurred on dip slopes with groundwater infiltration. Therefore, counterweight fill on the toe slope is proposed in this chapter to change the mode of failure from toe sliding to thrust failure.

Two series of centrifugal modeling were performed: one with a slope angle of 30 degrees and a reverse toe angle of 0 degrees, and the other with a slope angle of 30 degrees and a reverse toe angle of 15 degrees. The first slope series was mitigated using two different geometries (type 1 and type 2) of counterweight fill. Conversely, the second series was mitigated using the same type of slope but with different heights of counterweight fill.

Theoretical calculations of slope stability are provided in Section 6.1. The experimental setup and methodology are explained in Sections 6.2 and 6.3, respectively. In Section 6.4, experimental results are reported. Considering the importance of the reduction ratio for slope stability calculations, the reduction ratio was calculated and reported in Section 6.5. PIV analysis was employed to confirm the mode of failure in Section 6.6. In Section 6.7, the factor of safety was calculated and compared based on predictions and experimental results. Additionally, back-calculation methods were used to enhance the accuracy of the calculations by comparing them with experimental results. Furthermore, the optimal height of counterweight fill in the second series is discussed in this section. Finally, the findings from this chapter are summarized in Section 6.8.

6.1 Theoretical calculations of counterweight(berm) mitigation on slope lying bedding plane with lateral and toe supports with groundwater seepage

In this chapter, a counterweight berm is utilized to mitigate slope instability. The concept behind the berm is to enhance stability at the toe of the slope. The weight Q acts on the toe part, increasing the normal reaction force on the toe bedding. Two types of berms are proposed for slope mitigation: one for a slope angle of 30 degrees and a toe

angle of 0 degrees, as shown in Figure 6-1 and Figure 6-2. For a slope angle of 30 degrees and a toe angle of 15 degrees, a triangular berm with varying heights is added to the slope, as illustrated in Figure 6-3.

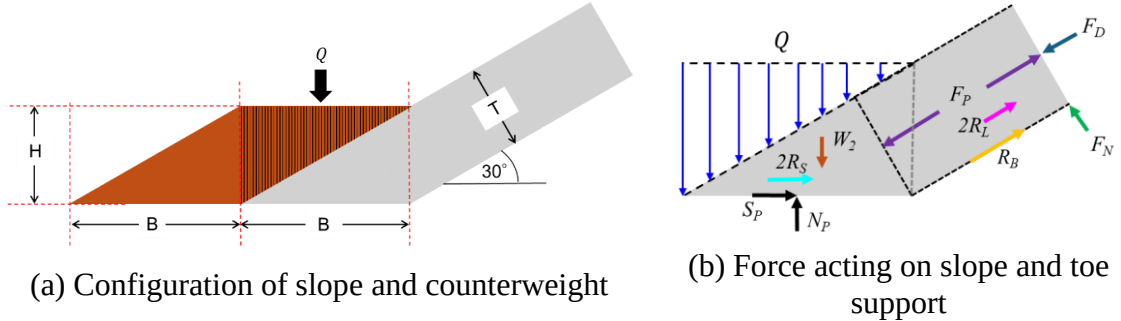


Figure 6-1 Idealization of mitigating counterweight (berm type 1) for slope 30° and toe 0°

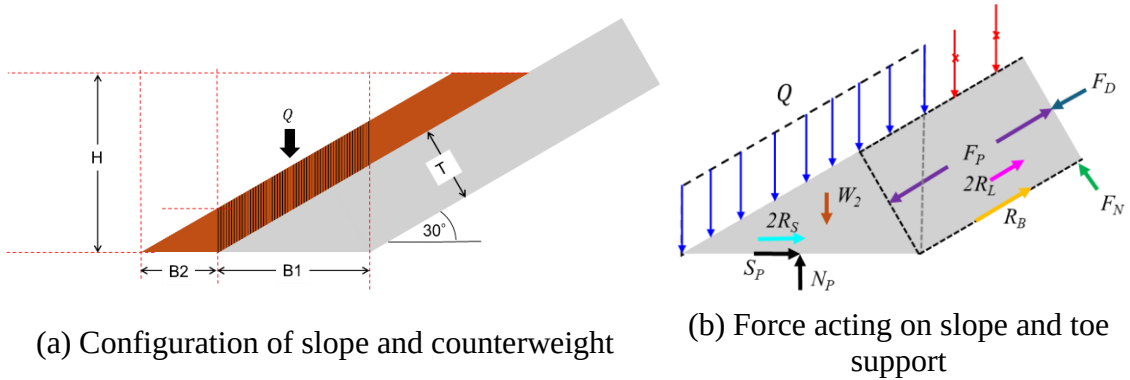


Figure 6-2 Idealization of mitigating counterweight (berm type 2) for slope 30° and toe 0°

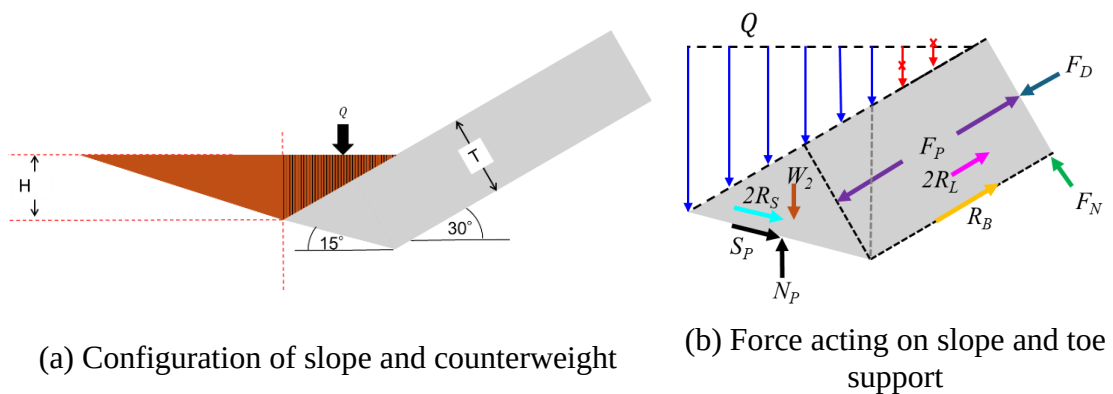


Figure 6-3 Idealization of mitigating counterweight (various berm height) for slope 30° and toe 15°

For the calculations, the concepts from the previous chapter are applied, with the addition of the weight Q incorporated into the equation. The forces acting on the triangular block are illustrated in Figure 6-1 to 6-3. F_p is the force triggering the movement of the triangular block. W_s represents the soil weight of the triangular block, R_s is the lateral resistance force, and Q is the weight of the berm. At the toe bedding plane, the normal force reaction (N_p) and the tangential reaction (S_p) are determined as shown in Eqs. (6-1) and (6-2).

$$N_p = F_p \sin(\alpha + \beta) + (W_2 + Q) \cos \beta \quad (6-1)$$

$$S_p = F_p \cos(\alpha + \beta) - (W_2 + Q) \sin \beta - 2R_2 \quad (6-2)$$

At toe bedding, the resistance force can be calculated based on Mohr-Coulomb failure criterion. That means if S_p is more than toe bedding resistance force, it has a potential to occur toe sliding. On the other hand, S_p less than resistance force, passive thrust failure can occur. Thus, S_p is limited by regrading resistance using following inequality, where ϕ_t is the interface friction angle, and c_t is the adhesion between the slope and toe support.

$$S_p \leq N_p \tan(\phi_t) + c_t BT \csc(\alpha + \beta) \quad (6-3)$$

To assess the limit of F_p on the virtual plane against sliding along the toe support, substituting Eqs. (6-1) and (6-2) into inequality (6-3) results in the following inequality after rearrangement:

$$\begin{aligned} & F_p \cos(\alpha + \beta + \phi_t) \sec \phi_t \\ & \leq c_t BT \csc(\alpha + \beta) + 2R_s - (W_2 + Q) \sin(\beta + \phi_t) \sec \phi_t \end{aligned} \quad (6-4)$$

If $\cos(\alpha + \beta + \phi_t) \leq 0$ or $(\alpha + \beta) \geq \pi/2 - \phi_t$ then inequality (6-4) is valid because the right-hand side of the equation is always positive within a normal range as compression ensures $FP > 0$. However, if $\cos(\alpha + \beta + \phi_t) \geq 0$ or $(\alpha + \beta) \leq \pi/2 - \phi_t$, then the limit $F_p \leq P_s$ can be thoroughly derived from inequality (6-4), where the sliding thrust P_s is determined by Eq. (6-5) through algebraic manipulation.

$$P_s = \frac{(c_t BT \csc(\alpha + \beta) + 2R_s) \cos(\phi_t) + (W_2 + Q) \sin(\beta + \phi_t)}{\cos(\alpha + \beta + \phi_t)} \quad (6-5)$$

It should be noted that $P_s > 0$ indicates the threshold for reverse toe sliding, while $P_s < 0$ suggests no sliding occurs on the toe support and R_s acts in the opposite direction, as shown in Fig. 6-1. The compressive thrust F_p must not surpass the limits set by either the passive thrust P_T or the sliding thrust P_s , provided that $FS > 0$. Consequently, thrust

resistance R_P is defined as P_T if P_S is negative; otherwise, it should be the positive minimum between P_S and P_T , as expressed in the following conditional equation:

$$R_P = \begin{cases} P_T & \text{if } P_S < 0 \\ \min\{P_S, P_T\} & \text{otherwise} \end{cases} \quad (6-6)$$

Then normal stress and shear stress underneath the triangular block are calculated by N_P and S_P based on the contact area with the toe support, as expressed in following equation:

$$\sigma = \frac{R_P \sin(\beta + \phi_t) + (W_2 + Q) \cos \beta}{BT \csc(\alpha + \beta)} \quad (6-7)$$

$$\tau = \frac{R_P \cos(\alpha + \beta) - (W_2 + Q) \sin \beta - 2R_S}{BT \csc(\alpha + \beta)} \quad (6-8)$$

The total resistance is composed of three components: bedding resistance (R_B), lateral resistance (R_L) from both side and toe resistance (R_P). This relationship is expressed by the following equation.

$$R_T = R_P + R_B + 2R_L \quad (6-9)$$

Therefore, slope stability can be calculated and represented as factor of safety as determined by Eq. (5-24)

$$FS = \frac{R_T}{F_D} \quad (6-10)$$

The concept of using a berm for mitigation is to change the mode of failure from toe sliding to thrust failure. It is necessary to define the optimum berm height because if the berm height is sufficient to stabilize the slope, adding more height is unnecessary. The optimum counterweight Q_P is calculated using Eq. (6-10), as shown in Eq. (6-11). Moreover, the optimum berm height can be divided into Eq. (6-12)

$$Q_P = \frac{(P_T \cos(\alpha + \beta + \phi_t)) - (c_i BT \csc(\alpha + \beta) + 2R_S) \cos \phi_t}{\sin(\beta + \phi_t)} - W_2 \quad (6-11)$$

$$z = T \csc(\alpha + \beta) \cos \beta \tan \alpha \quad (6-12)$$

6.2 Experimental setup and preparation of centrifuge physical model

The experiments comprised five cases, divided into two series. The first series involved a slope angle of 30 degrees with a reverse toe angle of 0 degrees (see Figure 6-4(a)). In this series, two cases were examined, each featuring a different shape of berm. Berm type 1 had dimensions of 3.5 cm in height, 6 cm in length, and 40 cm in width (see Figure 6-1), while berm type 2 had dimensions of 7 cm in height, 3 cm in length, and 40 cm in width (see Figure 6-2). The second series included a slope angle of 30 degrees with a reverse toe angle of 15 degrees, comprising three cases (see Figure 6-4(b)). In this series, all berms shared the same shape (see Figure 6-3) but varied in height, with measurements of 1 cm, 1.75 cm, and 2 cm.

During the experimental procedure, slope movement was induced by two factors: the surcharge weight, which acted as a moving boundary to exert pressure on the slope in the model test, and watering, simulating groundwater seepage. As depicted in the schematic diagram in Figure 6-4, the surcharge load cart was controlled by a motor, while water supplementation was regulated by a solenoid valve connected to the top supply tank.

The experimental material utilized in this study was Hiroshima sand, sourced from Kure Shi, Hiroshima prefecture. Initial processing involved sieving the sand to a diameter of 1 mm. Subsequently, the sand was adjusted to a water content of 8% before compaction. Compaction was conducted to achieve a compaction degree of 90%. The basic properties of the Hiroshima sand are summarized in Table 6-1.

To determine the interface shear strength, tilting tests were conducted. Given the potential for soil sliding along the bedding plane, as well as the presence of lateral and toe supports, Teflon sheets were employed as the bedding plane, while aluminum materials were utilized for the lateral and toe supports in the experimental setup.

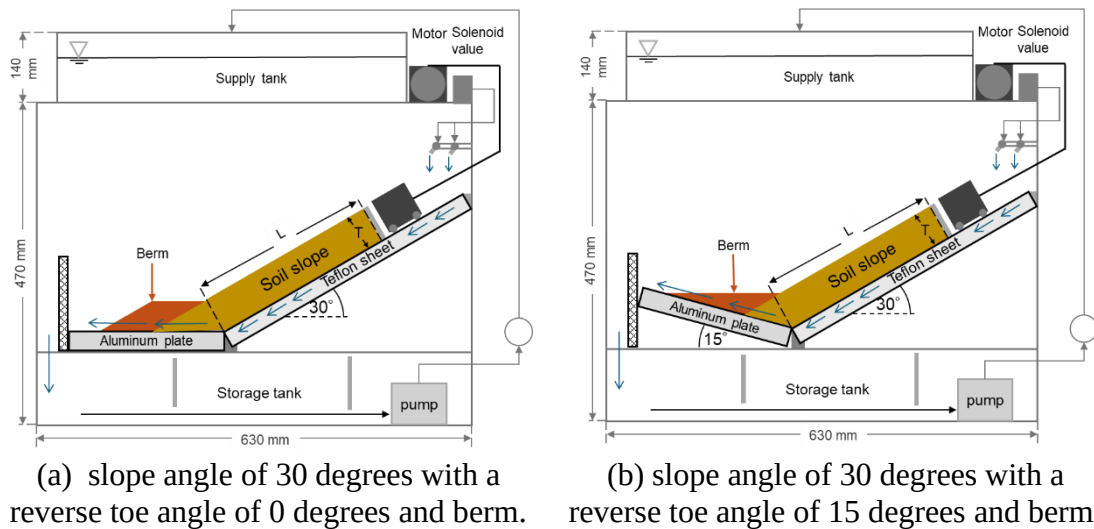


Figure 6-4 Schematic diagram of triggering for centrifuge model

Table 6-1 Basic properties of Hiroshima sand

Dry maximum density (ρ_{dmax})	1.747 g/cm ³
Water content (w)	8.0%
Degree of compaction (D_c)	90%
Coefficient of permeability (k)	2.4×10^{-5} m/s
Friction angle (ϕ)	33.13°
Apparent cohesion (c)	0.309 kPa
Interface friction angle of humid sand on Aluminum(ϕ_i)	25.4°
Interface adhesion of humid sand on Aluminum (c_i)	0.22 kPa
Interface friction angle of humid sand on Teflon sheet (ϕ_t)	29.0°
Interface adhesion of humid sand on Teflon sheet (c_t)	0.006 kPa

6.2.1 Instrumentation

The test conditions regarding the geometry are elaborated in Table 6-2. For monitoring purposes, seven pore water pressure sensors and nine earth pressure gauges were utilized and positioned according to the configuration illustrated in Figure 6-5. Earth pressure sensors were positioned on the bedding plane and lateral support near the bedding plane. Meanwhile, pore water pressure transducers were placed on the bedding to observe changes in seepage during water supplementation. Additionally, three laser sensors were deployed to track soil movement throughout the experiments. The first laser sensor was directed towards the surcharge to detect surcharge movement during its application. The second laser sensor was aimed at the middle slope to observe slope upheaval during slope collapse. Lastly, the third laser sensor was directed towards the berm near the toe support to observe berm movement during slope collapse. Figure 6-6 illustrates the laser sensors' orientations in each experiment series.

Table 6-2 Geometries of slope model lying bedding plane with lateral, toe supports and various berm types.

Test no.	Slope angle, α (°)	Toe angle, β (°)	Thickness (cm)	Width (cm)	Length (cm)	Surcharge weight(kg)	n_g	Berm
1	30	0	3	40	20	4.5	50	Type1
2	30	0	3	40	20	4.5	50	Type2
3	30	15	3	40	20	4.5	50	1.00 cm
4	30	15	1	40	20	4.5	50	1.75 cm
5	30	15	3	40	20	4.7	50	2.00 cm

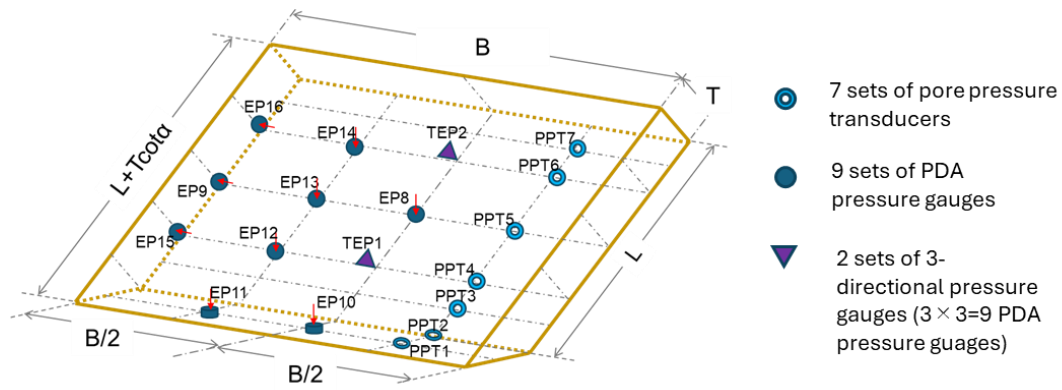


Figure 6-5 The instrumentation set up

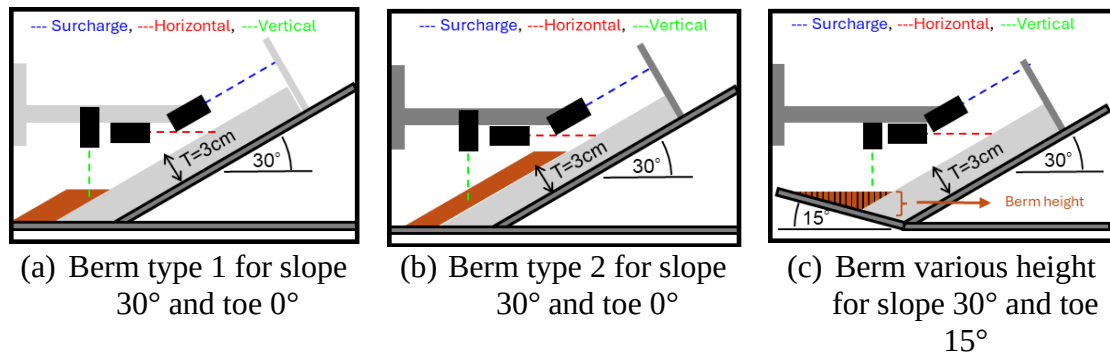


Figure 6-6 Pointing position of laser sensors

6.2.2 Preparation of centrifuge physical model

The preparation of the physical model began with adjusting the Hiroshima sand to achieve a water content of 8% and a specific weight. Following this, pore water pressure transducers and earth pressure sensors were meticulously installed on the Teflon sheet (bedding) and aluminum plate (lateral support), as depicted in Figure 6-7. Subsequently, the soil chamber underwent rotation by a crane to orient the bedding plane horizontally, facilitating compaction.

Once the bedding plane was properly aligned, a wooden plate was affixed to confine the model to the desired length of the slope. The adjusted Hiroshima sand was then carefully placed into the prepared mold, as shown in Figure 6-8, and compacted within delineated lines to control the compaction degree. Upon completion of compaction, the soil chamber was rotated back to its original position, and the berm was constructed. For the first series (slope 30° and toe 0°), berm type 1 was constructed using a restricted mold, then compacted soil was cut to shape, as illustrated in Figure 6-9. However, for berm type 2, a berm of 4.5 cm height with same length of slope was constructed first and then cut into the desired shape, as shown in Figure 6-10. For the second series (slope 30° and toe 15°), the slope was constructed first, and then compacted

berm with various heights was added later, as depicted in Figure 6-11. After the physical model was constructed, laser sensors were installed according to Figure 6-6.

Subsequently, the surcharge load cart and water supplementation apparatus were installed to induce slope failure. A top water tank was then positioned to cover the soil chamber and securely fastened with nuts. Following these preparations, the soil chamber was weighed and installed into the centrifuge pit for experimentation.

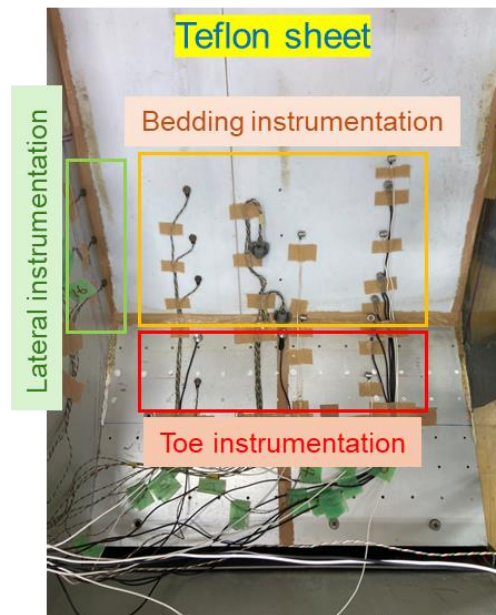


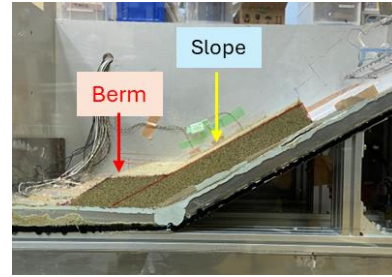
Figure 6-7 Installation of instruments



Figure 6-8 Rotation of soil chamber and compaction



(a) Compaction and cutting berm near toe

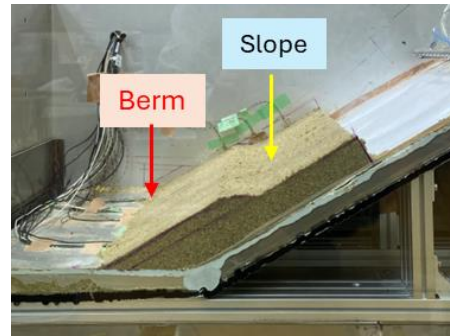


(b) After cutting

Figure 6-9 Construction of berm type 1 on slope 30° and toe 0°



(a) Compaction and cutting berm near toe



(b) After cutting

Figure 6-10 Construction of berm type 2 on slope 30° and toe 0°

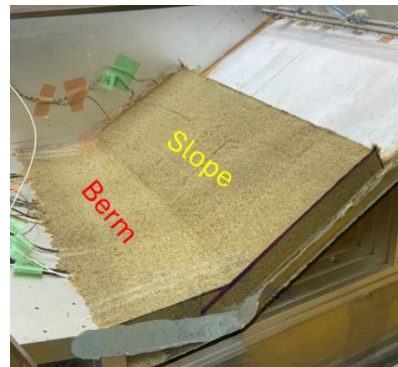


Figure 6-11 Construction of berm on slope 30° and toe 15°

6.3 Method

The experimental procedure is shown in Figure 6-12. Once the soil model was installed in the centrifuge room and connected to the data logger, centrifuge acceleration was increased to the target of 50G and maintained. The slope model was then triggered to fail using a combination of surcharge and water. However, the sequence of adding surcharge and water could vary. If water was added first without triggering slope failure, surcharge was subsequently added, and the load cell recorded the applied force on the slope model during surcharge release. Conversely, if surcharge was added first and slope failure did not occur, water was then added to induce slope failure. If slope failure

occurred after adding surcharge first, the experiment was conducted under dry conditions, with surcharge triggering the failure. During slope failure, high-speed cameras were employed to record the movement of the slope from a lateral perspective.

Five experiments were conducted under the following conditions:

Test 1: 50G, watering, surcharge

Test 2: 50G, watering, surcharge

Test 3: 50G, watering, surcharge

Test 4: 50G, watering, surcharge

Test 5: 50G, surcharge, watering

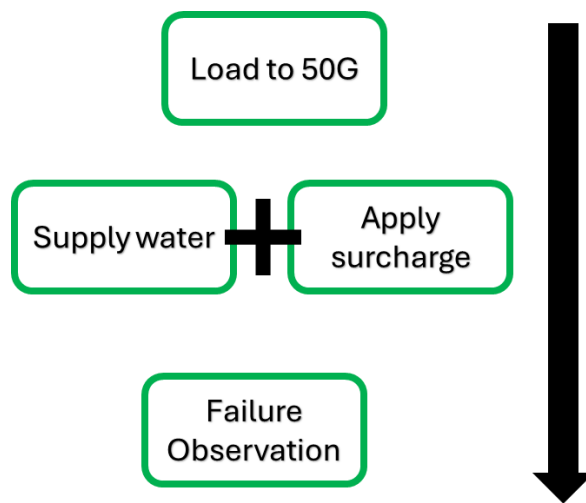


Figure 6-12 Experimental procedure

6.4 Experimental result

After completing the experiment, the data collected from various sensors including laser sensors (measuring displacement in mm), load cell (recording drag force in N), earth pressure sensors (measuring in kPa), and pore water pressure sensors (measuring in kPa) were analyzed. The elapsed time was recorded and displayed during slope failure. The experimental results are comprehensively elucidated through the detailed analysis of Tests 1 to 5.

6.4.1 Test 1: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: Type 1

Test procedure: 50G, watering, surcharge.

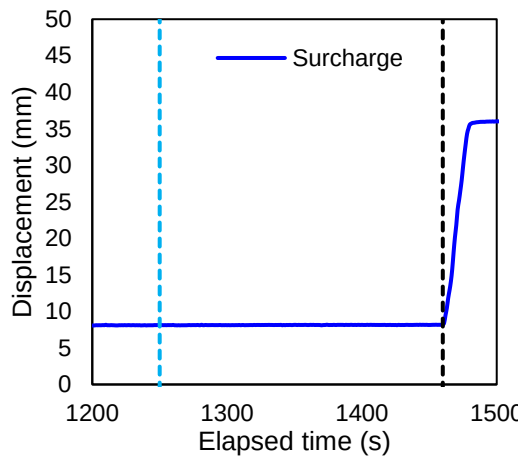
Displacement: During the experiment, two of the laser sensors did not register any activity. Only the laser sensor detecting the surcharge showed any response. While the surcharge remained stationary during water supplementation, movement was detected when the surcharge was applied to the slope.

Drag force: Once the centrifugal acceleration reached 50G, the load cell measured a drag force of approximately -442.86N. Upon applying the surcharge to the slope, the drag force immediately increased to 32.44N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge. Consequently, the applied force was calculated to be 475.3N.

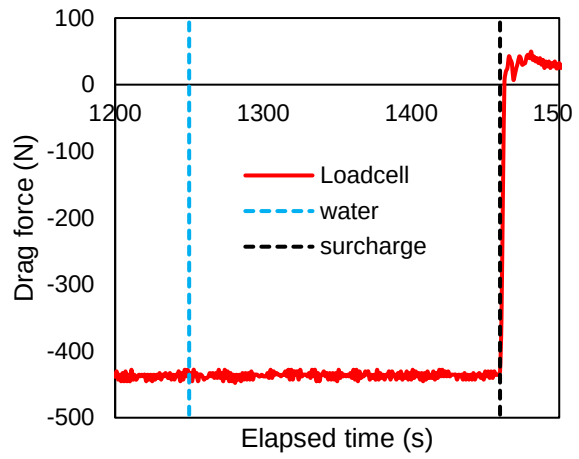
Earth pressure: During water supplementation, the earth pressure remained constant. However, after the surcharge was applied to the slope, all earth pressures increased, with EP16 showing a particularly notable increase.

Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, located near the toe slope, recorded a maximum pore pressure of 7.29 kPa. Additionally, the average pore pressure on the slope, represented by PPT4 and PPT6, was approximately 0.57 kPa. However, after the surcharge was applied, all pore pressures dropped.

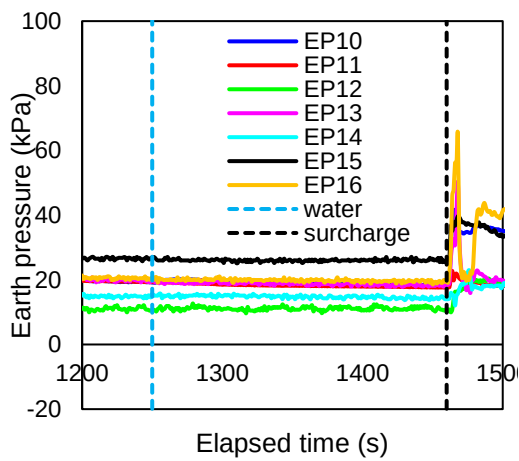
Figure 6-13 illustrates the response of instruments during the experimental procedure. In Figure 6-14, the slope failure following the completion of the centrifuge experiment is depicted. The movement predominantly occurred at the middle to crest of the slope, and upheaval was observed, as shown in Figure 6-14(b), indicating thrust failure. Additionally, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was symmetrical, as depicted in Figure 6-14(a).



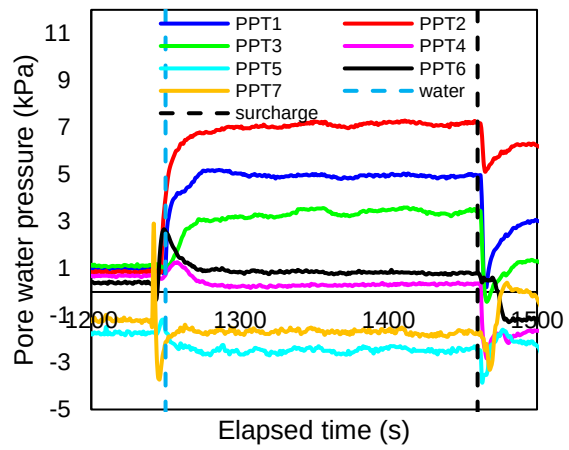
(a) Slope displacement during slope failure



(b) Drag force during slope failure

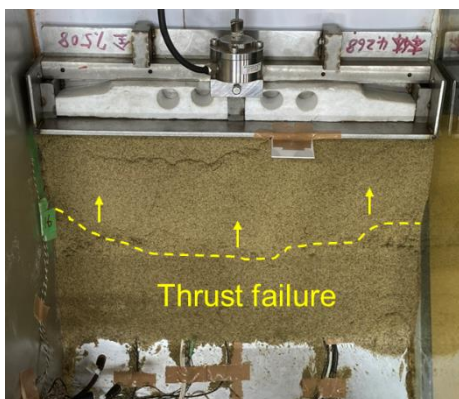


(c) Earth pressure during slope failure



(d) Pore water pressure during slope failure

Figure 6-13 Experimental result of test 1



(a) Top view of slope



(b) Side view of slope

Figure 6-14 Slope failure observed after the testing of Test1

6.4.2 Test 2: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: Type 2

Test procedure: 50G, watering, surcharge.

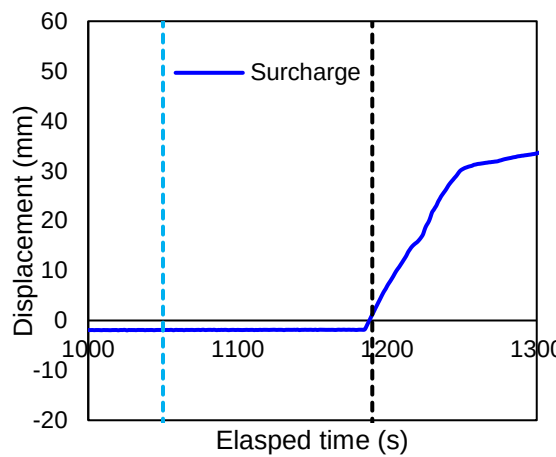
Displacement: During the experiment, two of the laser sensors did not register any activity. Only the laser sensor detecting the surcharge showed any response. While the surcharge remained stationary during water supplementation, movement was detected when the surcharge was applied to the slope.

Drag force: Once the centrifugal acceleration reached 50G, the load cell measured a drag force of approximately -427.94N. Upon applying the surcharge to the slope, the drag force immediately increased to 67.18N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge. Consequently, the applied force was calculated to be 495.12N.

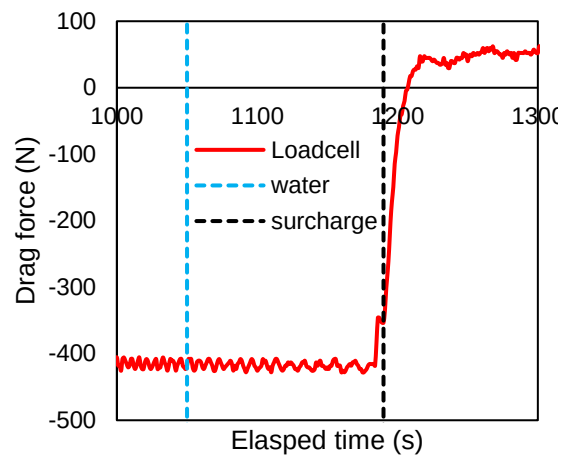
Earth pressure: During water supplementation, the earth pressure remained constant. However, after the surcharge was applied to the slope, all earth pressures increased, with EP16 showing a particularly notable increase.

Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, located near the toe slope, recorded a maximum pore pressure of 4.85kPa. Additionally, the average pore pressure on the slope, represented by PPT4 and PPT6, was approximately 1.58kPa. However, after the surcharge was applied, all pore pressures dropped.

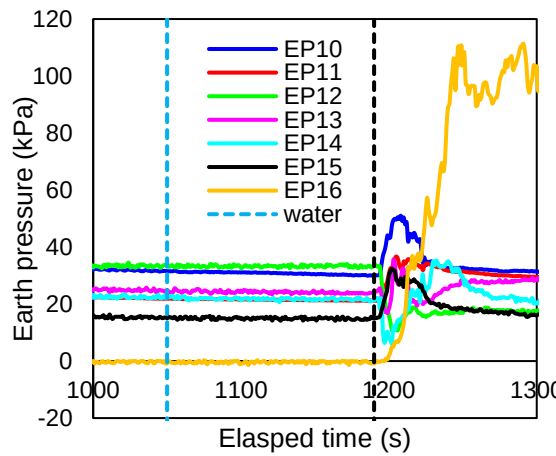
Figure 6-15 illustrates the response of instruments during the experimental procedure. In Figure 6-16, the slope failure following the completion of the centrifuge experiment is depicted. The movement predominantly occurred at the middle to crest of the slope, and upheaval was observed, as shown in Figure 6-16(b), indicating thrust failure. Additionally, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was unsymmetrical, as depicted in Figure 6-16(a).



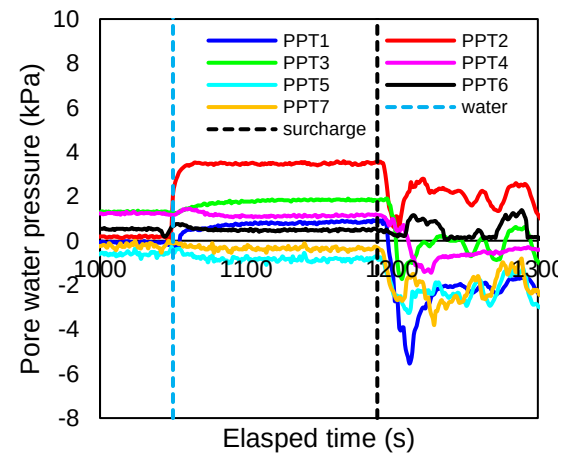
(a) Slope displacement during slope failure



(b) Drag force during slope failure

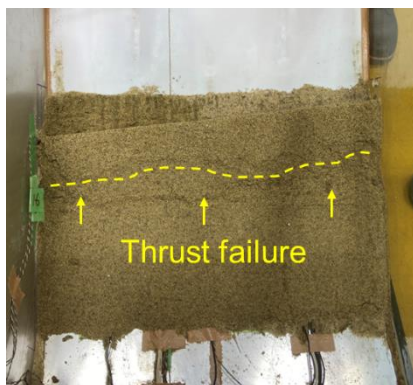


(c) Earth pressure during slope failure



(d) Pore water pressure during slope failure

Figure 6-15 Experimental result of test 2



(a) Top view of slope



(b) Side view of slope

Figure 6-16 Slope failure observed after the testing of Test2

6.4.3 Test 3: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 1cm of height

Test procedure: 50G, watering, surcharge.

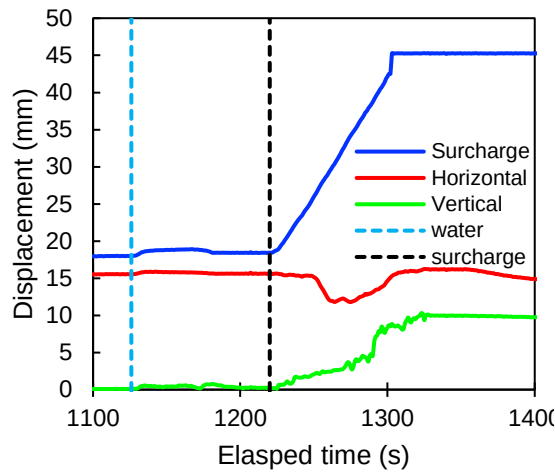
Displacement: Upon reaching a centrifugal acceleration of 50G, there was observed movement in the displacement of the surcharge and horizontally. However, during water supplementation, the displacement from laser sensors remained stable. Subsequently, upon applying the surcharge, there was a significant increase in vertical displacement on the berm (surcharge displacement increased), indicating toe sliding. However, horizontal displacement on the middle slope fluctuated during the application of the surcharge.

Drag force: Once the centrifugal acceleration reached 50G, the load cell measured a drag force of approximately -345.83N. Upon applying the surcharge to the slope, the drag force immediately increased to -47.27N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge. Consequently, the applied force was calculated to be 298.56N.

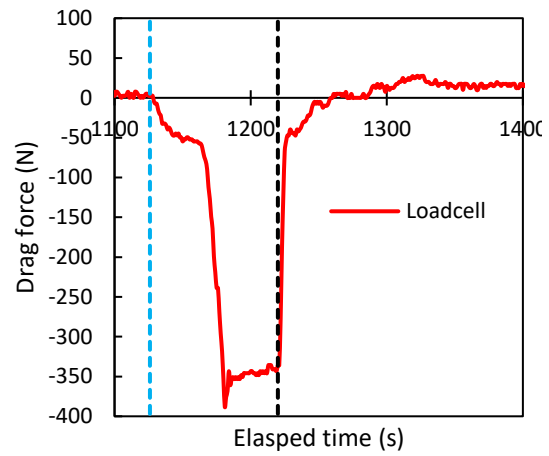
Earth pressure: During water supplementation, the earth pressure remained constant. However, after the surcharge was applied to the slope, all earth pressures increased, with EP10 and EP15 showing a particularly notable increase.

Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, located near the toe slope, recorded a maximum pore pressure of 9.84kPa. Additionally, the average pore pressure on the slope, represented by PPT4 and PPT6, was approximately 0.63kPa. However, after the surcharge was applied, all pore pressures dropped.

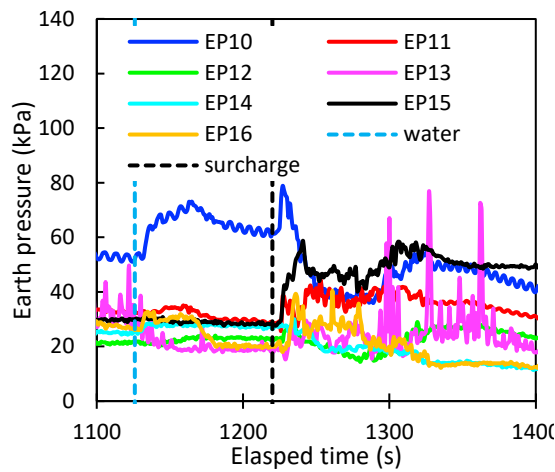
Figure 6-17 illustrates the response of instruments during the experimental procedure. In Figure 6-18, the slope failure following the completion of the centrifuge experiment is depicted. The movement predominantly occurred at the berm located near the toe slope, and the berm's upheaval was observed, as shown in Figure 6-18(b), indicating toe sliding. This result aligns well with the displacement result, which indicates increased vertical displacement at the berm, as shown in Figure 6-17(a). Additionally, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was symmetrical, as depicted in Figure 6-18(a).



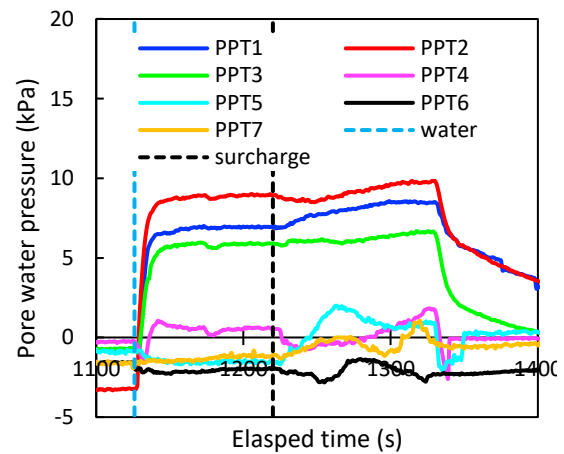
(a) Slope displacement during slope failure



(b) Drag force during slope failure



(c) Earth pressure during slope failure



(d) Pore water pressure during slope failure

Figure 6-17 Experimental result of test 3



(a) Top view of slope



(b) Side view of slope

Figure 6-18 Slope failure observed after the testing of Test3

6.4.4 Test 4: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 1.75cm of height

Test procedure: 50G, watering, surcharge.

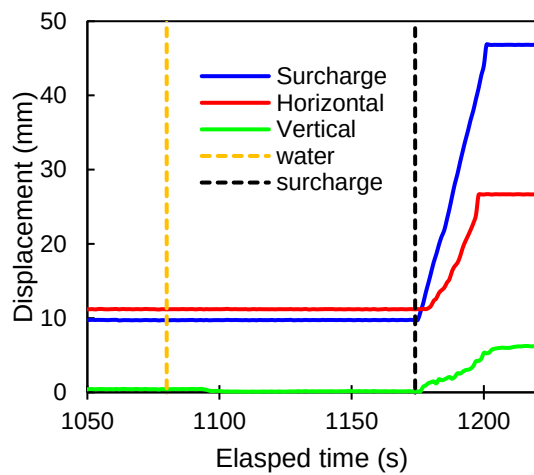
Displacement: Upon reaching a centrifugal acceleration of 50G, there was observed movement in the displacement of the surcharge and horizontally. However, during water supplementation, the displacement from laser sensors remained stable. Subsequently, upon applying the surcharge, there was a significant increase in horizontal and vertical displacement, indicating toe sliding and thrust failure.

Drag force: Once the centrifugal acceleration reached 50G, the load cell measured a drag force of approximately -490.14N. Upon applying the surcharge to the slope, the drag force immediately increased to 34.83N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge. Consequently, the applied force was calculated to be 524.97N.

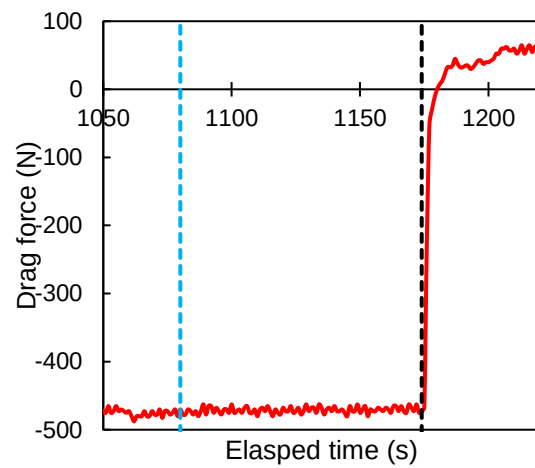
Earth pressure: During water supplementation, the earth pressure remained constant. However, after the surcharge was applied to the slope, all earth pressures increased, with EP12 and EP16 showing a particularly notable increase.

Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, located near the toe slope, recorded a maximum pore pressure of 14.63kPa. Additionally, the average pore pressure on the slope, represented by PPT4 and PPT6, was approximately 3.53kPa. However, after the surcharge was applied, all pore pressures dropped.

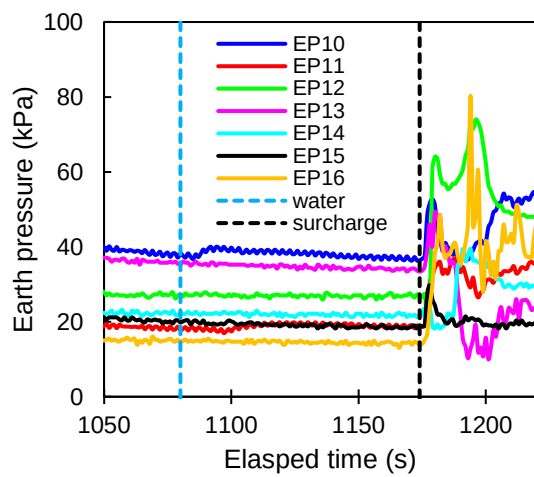
Figure 6-19 illustrates the response of instruments during the experimental procedure. In Figure 6-20, the slope failure following the completion of the centrifuge experiment is depicted. The movement predominantly occurred at the middle to crest of the slope, and upheaval was observed, as shown in Figure 6-20(b), indicating thrust failure. This result contradicts with the displacement result, which indicates increased vertical displacement at the berm, as shown in Figure 6-19(a). Thus, PIV analysis would be used to confirm slope movement during slope collapse. Additionally, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was symmetrical, as depicted Figure 6-20(a).



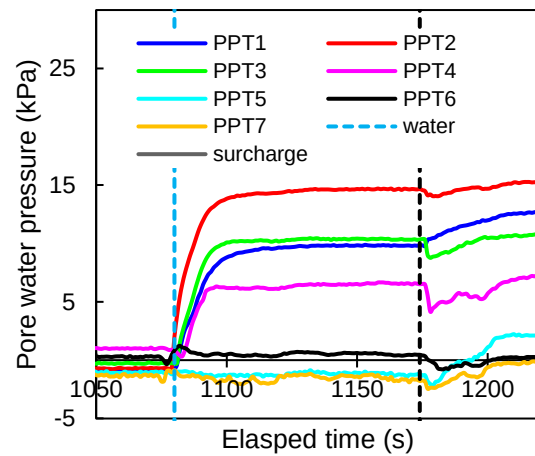
(a) Slope displacement during slope failure



(b) Drag force during slope failure



(c) Earth pressure during slope failure



(d) Pore water pressure during slope failure

Figure 6-19 Experimental result of test 4



(a) Top view of slope



(b) Side view of slope

Figure 6-20 Slope failure observed after the testing of Test4

6.4.5 Test 5: Geometry condition $\alpha=30^\circ$, $\beta=15^\circ$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 2cm of height

Test procedure: 50G, surcharge, watering.

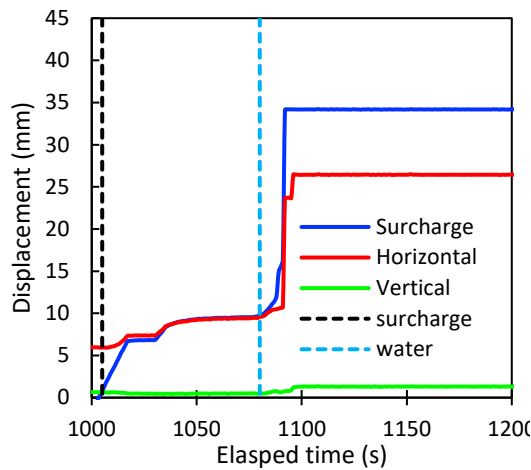
Displacement: Upon reaching a centrifugal acceleration of 50G, movement in the displacement of the surcharge and horizontally was observed. Surcharge was applied first and left onto the slope, resulting in increased displacement of the surcharge, as shown in Fig. 6-21(a). However, vertical displacement remained constant. There was a slight movement in horizontal displacement. During water supplementation, surcharge displacement immediately increased. Similarly, horizontal displacement increased. However, vertical displacement, which points to the berm, showed little change and remained stable, indicating thrust failure occurred in this case.

Drag force: Once the centrifugal acceleration reached 50G, the load cell measured a drag force of approximately -480.18N. Upon applying the surcharge to the slope, the drag force immediately increased to 44.78N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge. Consequently, the applied force was calculated to be 524.96N.

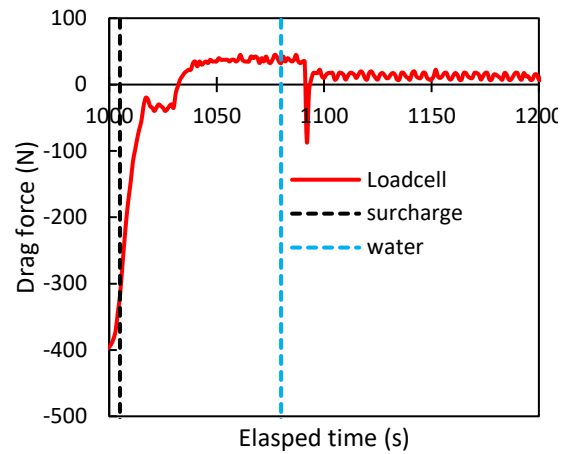
Earth pressure: During surcharge applying, all earth pressures increased and remained constant. After water supplementation, all earth pressures increased, with EP10 and EP16 showing a particularly notable increase.

Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, located near the toe slope, recorded a maximum pore pressure of 9.91kPa. Additionally, the average pore pressure on the slope, represented by PPT4 and PPT6, was approximately 1.786kPa.

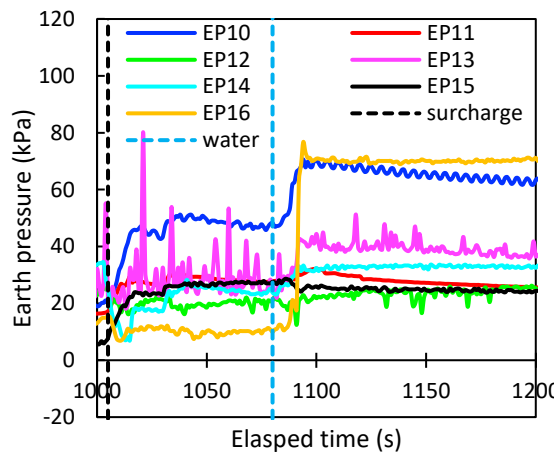
Figure 6-21 illustrates the response of instruments during the experimental procedure. In Figure 6-22, the slope failure following the completion of the centrifuge experiment is depicted. The movement predominantly occurred at the middle to crest of the slope, and upheaval was observed, as shown in Figure 6-22(b), indicating thrust failure. This result aligns well with the displacement result, which indicates constant vertical displacement at the berm, as shown in Figure 6-21(a). Additionally, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was symmetrical, as depicted in Figure 6-22(a).



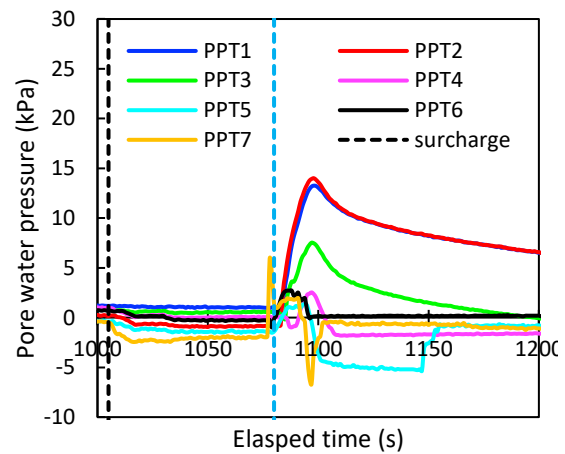
(a) Slope displacement during slope failure



(b) Drag force during slope failure

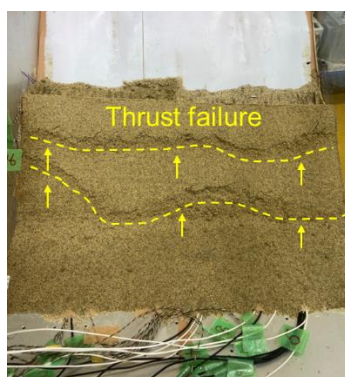


(c) Earth pressure during slope failure



(d) Pore water pressure during slope failure

Figure 6-21 Experimental result of test 5



(a) Top view of slope



(b) Side view of slope

Figure 6-22 Slope failure observed after the testing of Test5

6.5 Reduction factor

The experimental reduction ratio(r_d) is depicted in Figure 6-23. In Test 1, shown in Figure 6-24(a), during water supplementation, r_{d1} , r_{d2} and r_{d3} remained relatively unchanged. However, r_{d1} , r_{d2} and r_{d3} increased significantly upon the application of the surcharge, reaching a peak at 1.19. This value is crucial for subsequent calculations. In Test 2, during water supplementation, there was minimal change in r_{d1} , r_{d2} and r_{d3} . However, upon applying the surcharge to the slope, r_{d1} experienced a substantial increase, peaking at 0.90. Similarly, in Test 3, all r_d values flocculated and increased immediately upon adding the surcharge, with r_{d1} exhibiting a notable increase and reaching a peak at 0.87. For Test 4, All r_d remain negative value during water supplementation. All r_d increased immediately and peaked at 0.96 when surcharge added to slope. In Test 5, all r_d values increased immediately after adding the surcharge. During water supplementation r_{d3} significantly increased, peaking at 0.68.

The experimental results indicate an increase in the reduction ratio, peaking around 1, which aligns well with the calculation of passive earth pressure.

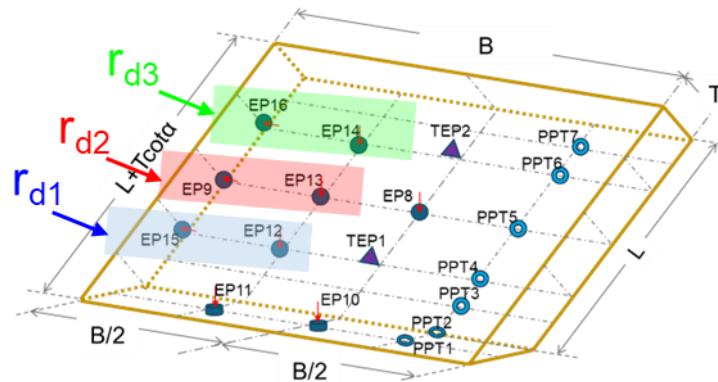


Figure 6-23 location of earth pressure gauge and pore water pressure sensors for calculating reduction ratio

6.6 PIV analysis

During the experiment's procedures, the slope collapsed rapidly, necessitating the deployment of a high-speed camera to document the event. This camera, operating at 100 frames per second and with a resolution of 1244x890 pixels, offered exceptional image quality. It captured images of the slope profile through the transparent side wall of the model chamber, focusing on the specific behavior of the slope model. This targeted approach ensured precise recording of critical details during the failure process. Consequently, the camera captured the slope's movement with remarkable clarity and fluidity, offering valuable insights into the dynamics of slope failure. The removal of the light reflection plate due to the significant size of the soil chamber led to some light being reflected onto the crest slope, as shown in Figure 6-25. Consequently, this reflection

affected the PIV analysis, resulting in its absence. PIV analysis was conducted on the recorded video footage capturing the slope collapse during each of the tests 1 to 5, as demonstrated in Figure 6-25 to 6-29.

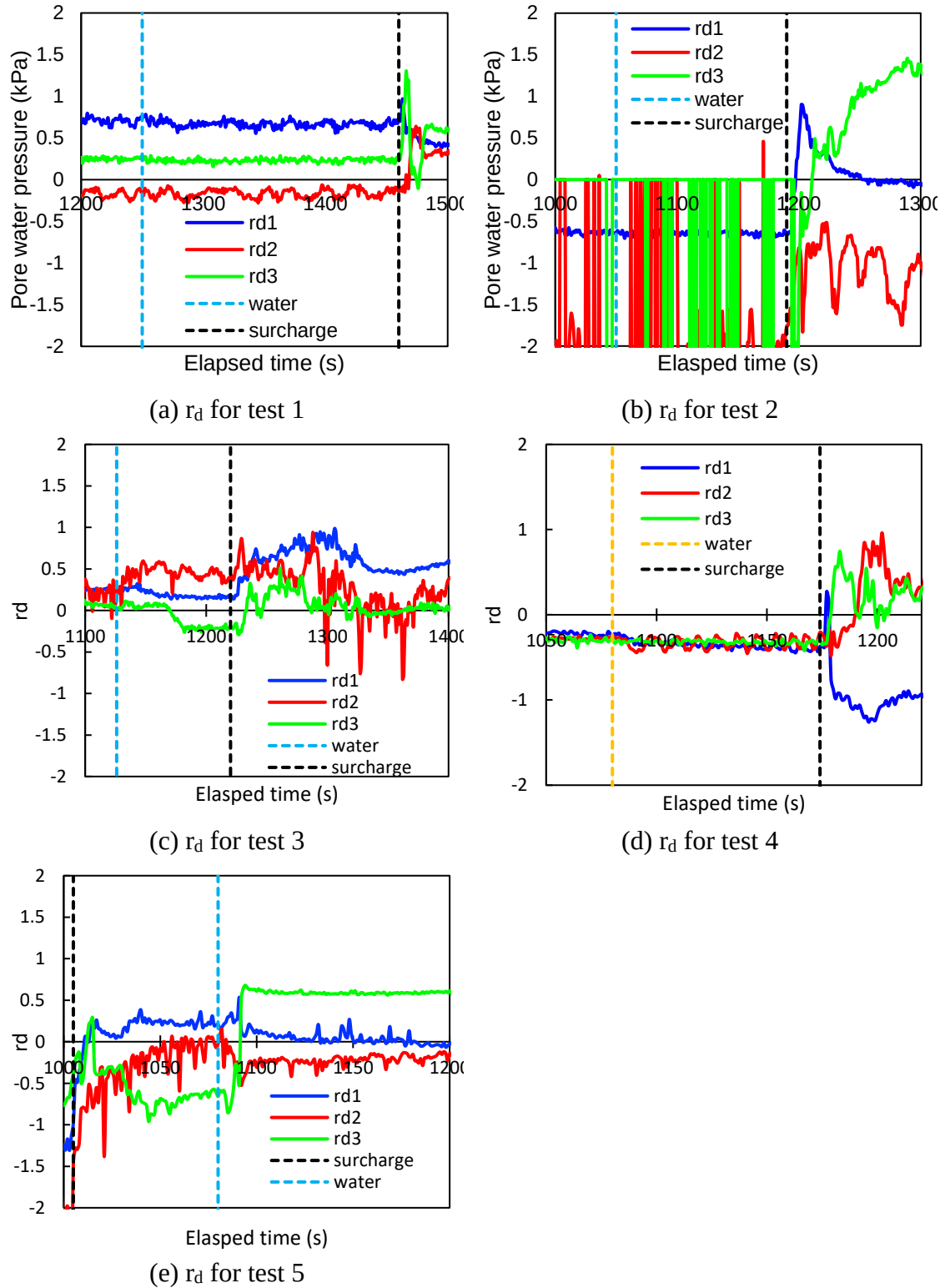


Figure 6-24 Experimental reduction ratio

Test 1: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Berm type 1

Test procedure: 50G, watering, surcharge.

Figure 6-25(a) displays an image captured during the slope collapse, with the displacement vector field visible. These displacement vectors are observed across the entire slope when the surcharge cart moved 1 cm, indicating toe sliding. However, during the 2 cm surcharge movement, thrust failure was observed, illustrated in Figure 6-25(b). This result confirms that berm type 1 can mitigate slope failure.

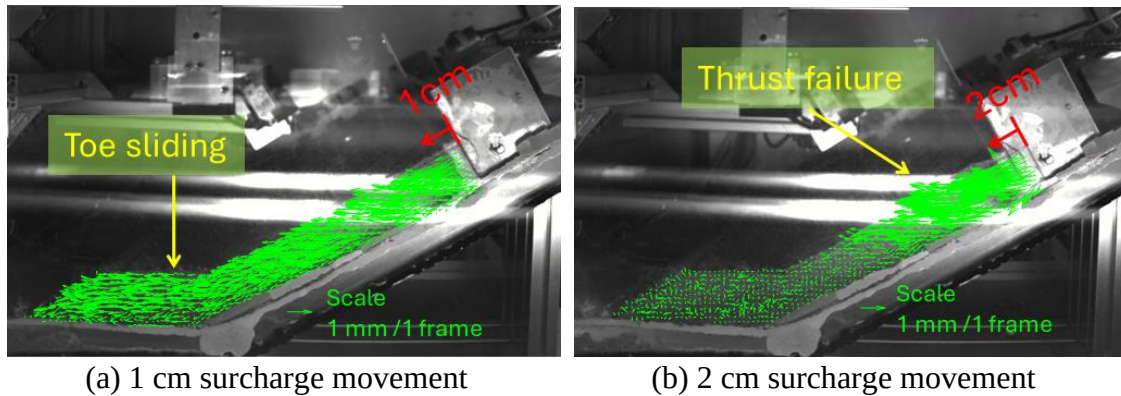


Figure 6-25 PIV analysis results of test 1

Test 2: Geometry condition $\alpha=30$, $\beta=0$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Berm type 2

Test procedure: 50G, watering, surcharge.

Figure 6-26(a) shows an image captured during the slope collapse, with the displacement vector field visible. These displacement vectors are observed across the entire slope when the surcharge cart moved 1 cm, indicating toe sliding. However, during the 2 cm surcharge movement, thrust failure was observed, shown in Figure 6-26(b). This result confirms that berm type 2 can mitigate slope failure.

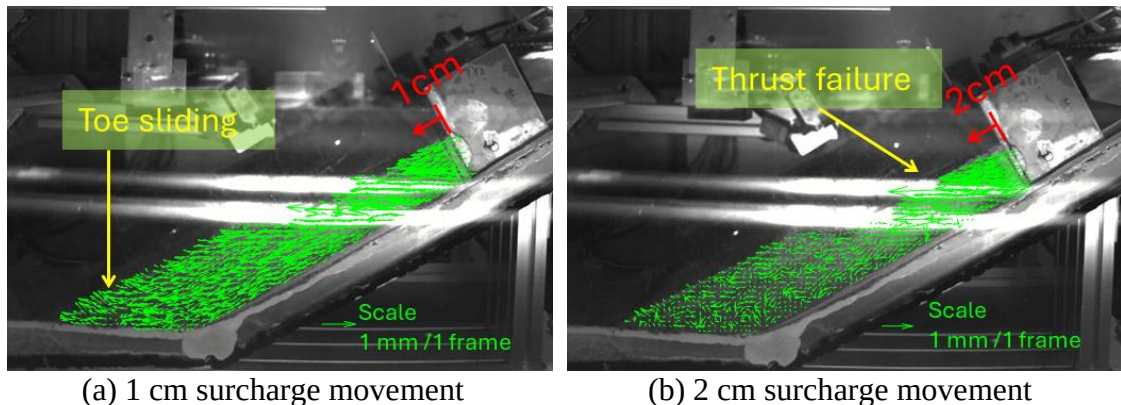


Figure 6-26 PIV analysis results of test 2

Test 3: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$
Berm height 1 cm

Test procedure: 50G, watering, surcharge.

Figure 6-27 shows an image captured during the slope collapse, with the displacement vector field visible. These displacement vectors are observed across the entire slope when the surcharge cart moved 2 cm, indicating toe sliding. This result confirms that a berm height of 1 cm does not have the potential to mitigate slope failure.

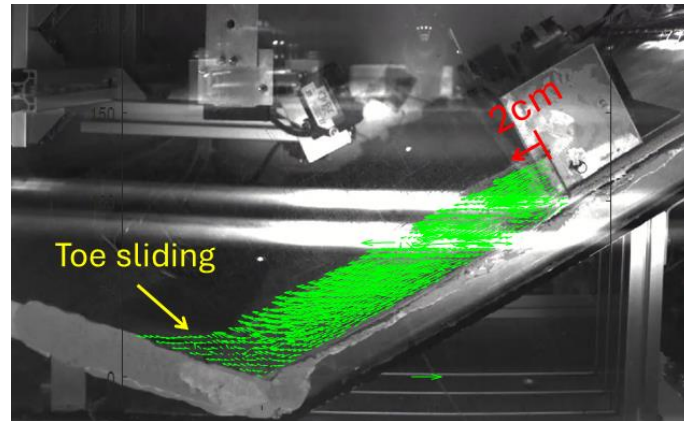


Figure 6-27 PIV analysis results of test 3

Test 4: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$
Berm height 1.75 cm

Test procedure: 50G, watering, surcharge.

Figure 6-28 shows an image captured during the slope collapse, with the displacement vector field visible. When the surcharge cart moved 2 cm, two distinct zones can be clearly identified from the PIV analysis results. The berm and toe slope areas showed no movement, while large displacement vectors were observed in the slope area. This result confirms that a berm height of 1.75 cm has the potential to mitigate slope failure.

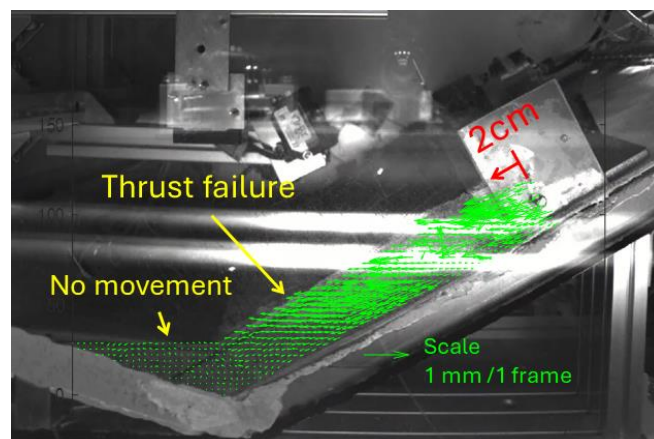


Figure 6-28 PIV analysis results of test 4

Test 5: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$
Berm height 2.0 cm

Test procedure: 50G, surcharge, watering.

Figure 6-29 shows an image captured during the slope collapse, with the displacement vector field visible. When the surcharge cart moved 2 cm, two distinct zones can be clearly identified from the PIV analysis results. The berm and toe slope areas showed no movement, while large displacement vectors were observed in the slope area. This result confirms that a berm height of 2.0 cm has the potential to mitigate slope failure. Additionally, it can be concluded that there is no significant benefit to increasing the berm height beyond 1.75 cm and 2.0 cm, as the mode of failure changes and the optimum height of the berm has been effectively determined.

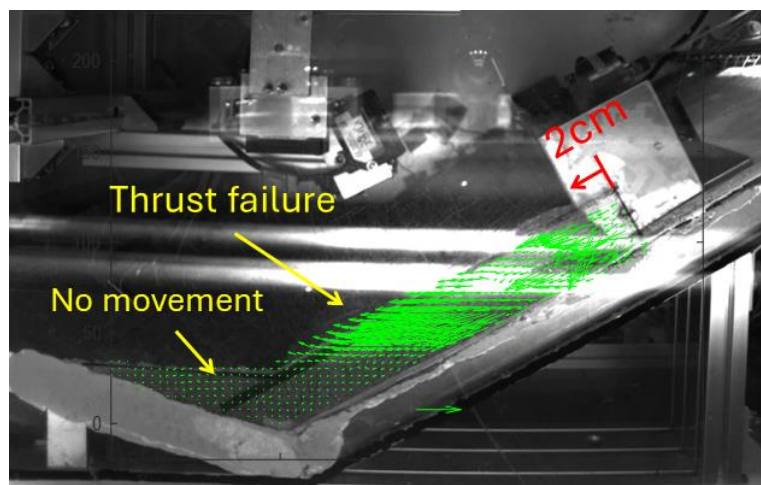


Figure 6-29 PIV analysis results of test 5

6.7 Discussion

6.7.1 Factor of safety (FOS)

The Factor of Safety (FOS) can be calculated based on the resistance force and driving force. However, in this chapter berm part increase stability on the part. In the experiment, pore-water pressure transducers and applied force measurements were recorded. These results can be analyzed in terms of FOS, as illustrated in Figure 6-30. To maximize lateral resistance and predict FOS, a reduction ratio (r_d) of +1 (passive) was assumed. Subsequently, the experimental r_d values were used for calculation and comparison with predictions. The predicted FOS in tests 2 to 4 indicates that slope collapse is likely to correspond with the displacement results obtained from laser sensors. However, in test 1 and test 2, the displacement data could not be compared, and the predicted FOS is close to 1.00, indicating slope instability.

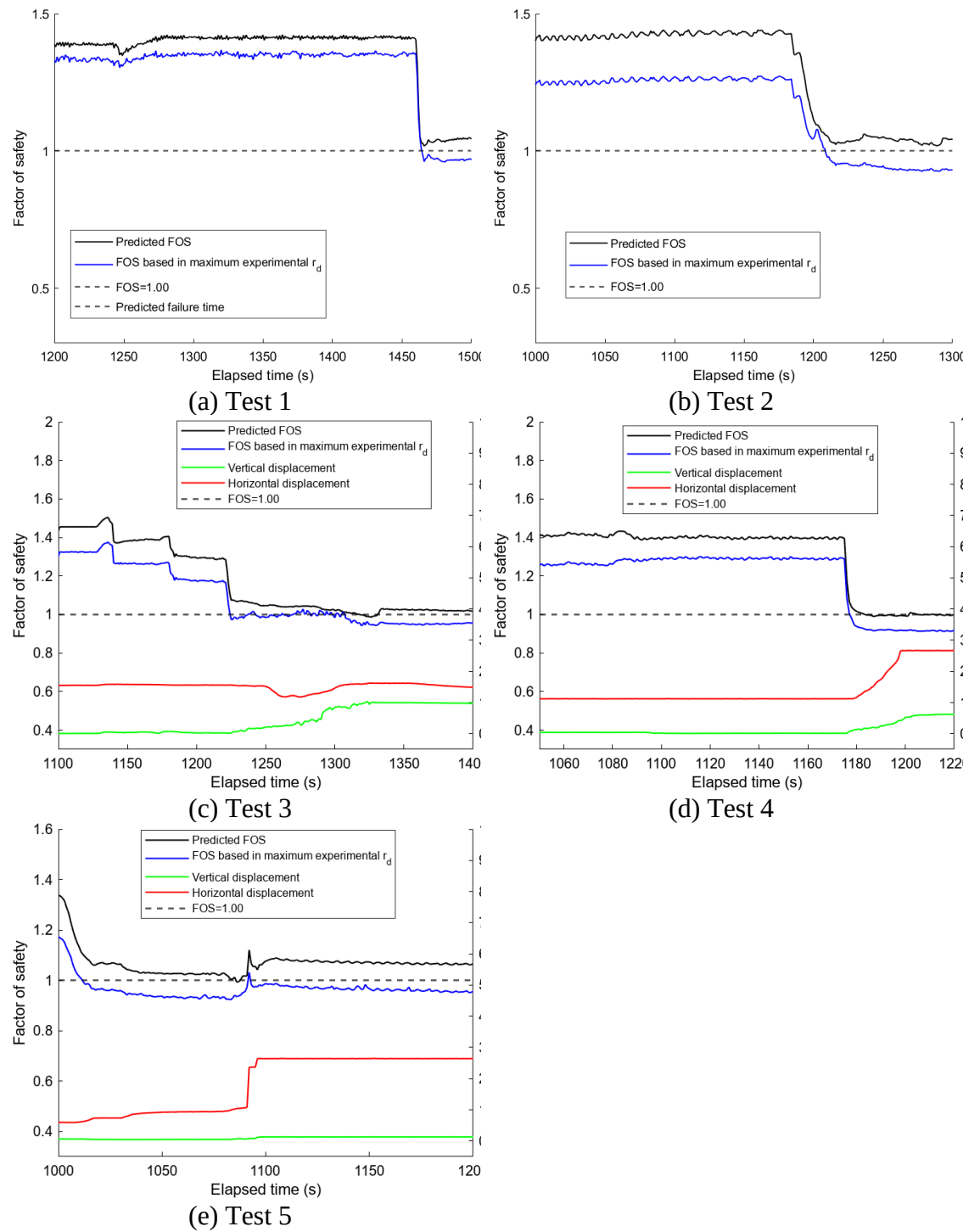


Figure 6-30 Comparison between prediction and experimental factor of safety

6.7.2 Failure mode

In Section 6.1, theoretical calculations are provided for failure mode analysis in Eq. (6-6). Toe resistance (P_s) and passive thrust limitation (P_T) can be used to determine the failure mode of the slope. Figure 6-31 illustrates the comparison between P_s and P_T ;

it can be seen that P_T is lower than P_S in the series with a slope angle of 30 degrees and a toe angle of 0 degrees, indicating that thrust failure occurs in both types of counterweights.

In the series with a slope angle of 30 degrees and a reverse toe angle of 15 degrees, test 3 (1 cm height) shows P_S is less than P_T , indicating toe sliding. However, tests 4 (1.75 cm height) and 5 (2 cm height) show P_T is more than P_S , indicating thrust failure. Thus, determining the optimum height of the counterweight is necessary.

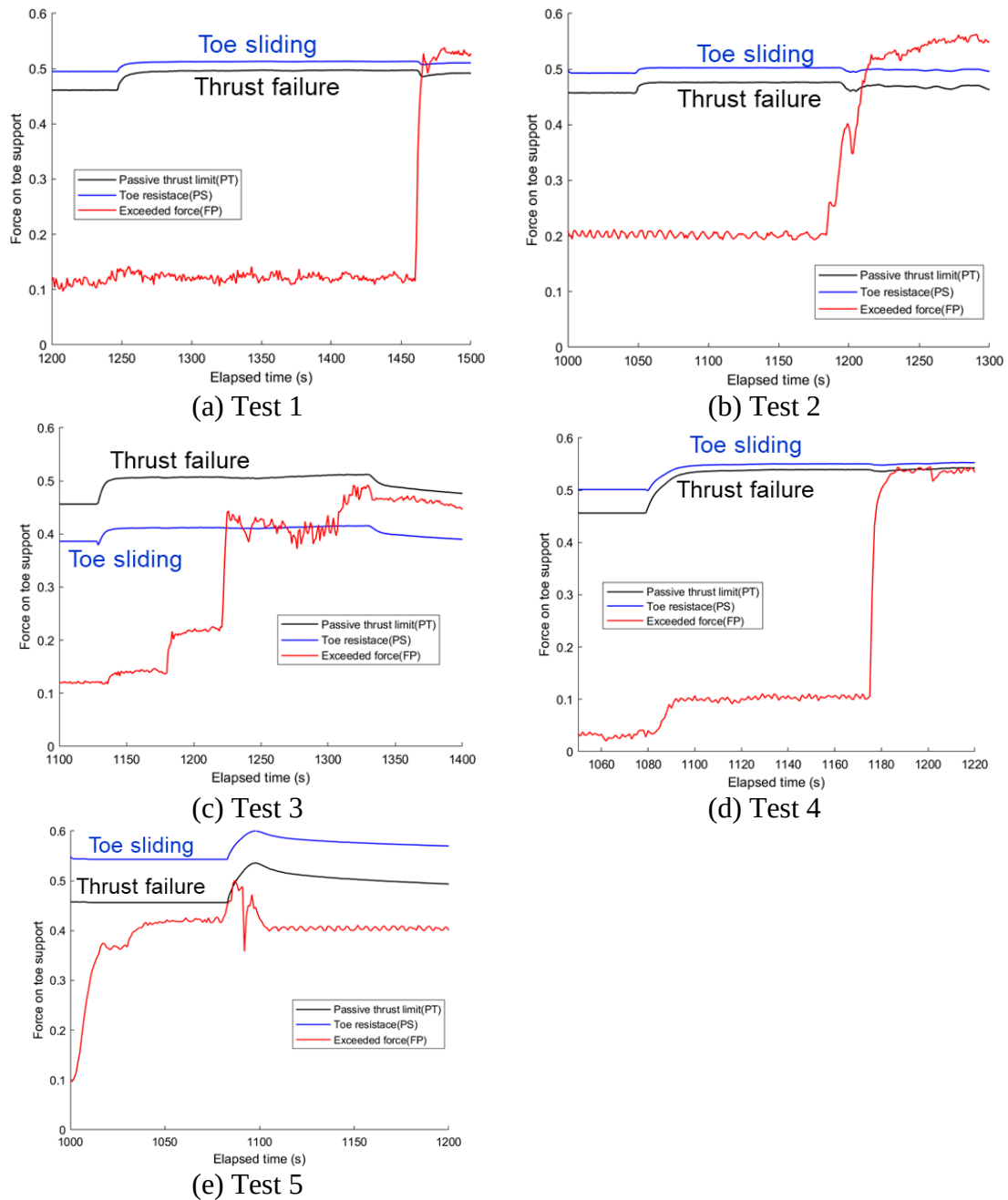


Figure 6-31 Comparison of toe resistance and passive thrust limitation

6.7.3 Optimum counterweight

Due to the counterweight fill in the series with a slope angle of 30 degrees and a reverse toe angle of 15 degrees, toe sliding occurred in test 3 (1 cm height), while thrust failure occurred in tests 4 (1.75 cm height) and 5 (2 cm height). The optimum counterweight height is therefore between 1 cm and 1.75 cm. Figure 6-32 illustrates the relationship between resistance and counterweight fill, where the intersection of PS and PT indicates the optimum counterweight fill, which is 0.065 kN or a counterweight height of 1.50 cm. Therefore, a 1.5 cm height of counterweight is sufficient to mitigate the dip slope with groundwater seepage.

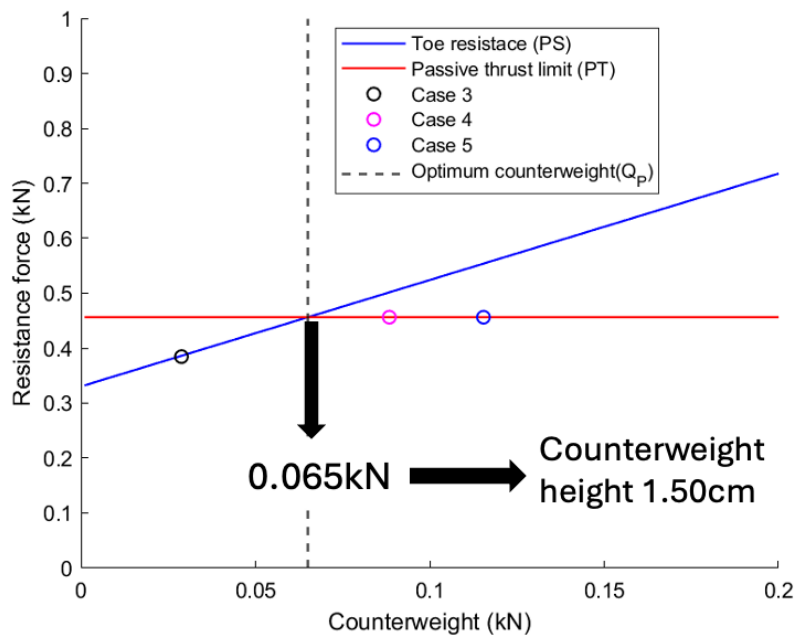
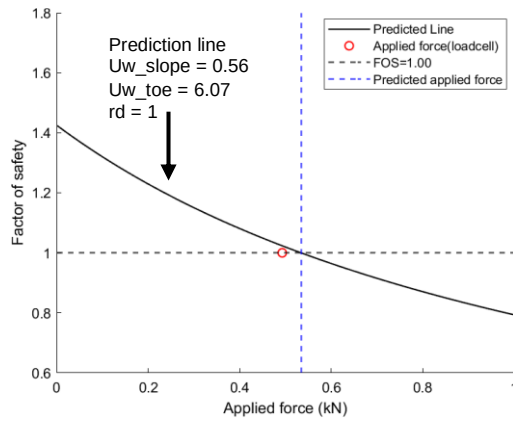


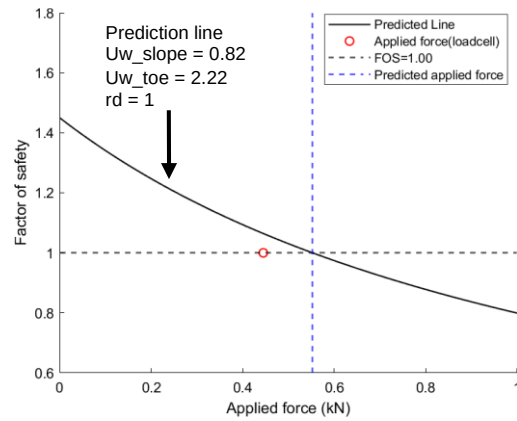
Figure 6-32 The relationship between resistance and counterweight fill of slope angle 30degree with reverse toe angle 15degree

6.7.4 Back calculation of applied force

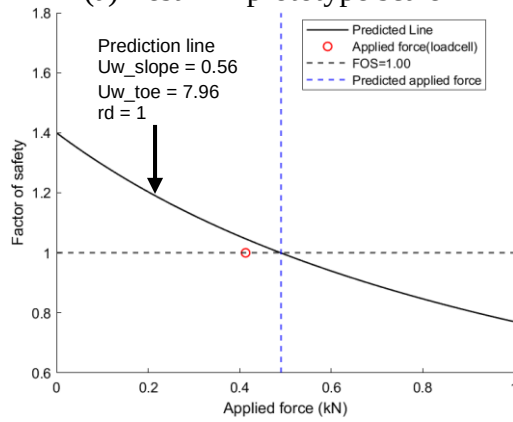
To enhance the accuracy of theoretical calculations, back-calculation of the applied force is of particular interest. Figure 6-33 illustrates the relationship between FOS and applied force. The prediction line represents slope stability considering existing pore-water pressure and $r_d = 1$. The applied force is increased until the FOS reaches 1.00, indicating slope failure. This predicted applied force is then compared with the applied force that led to slope failure in the experiments.



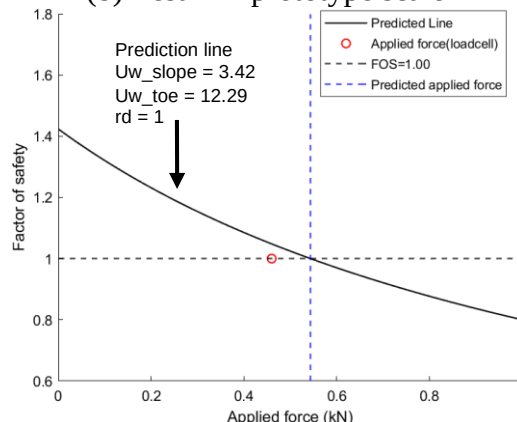
(a) Test 1 in prototype scale



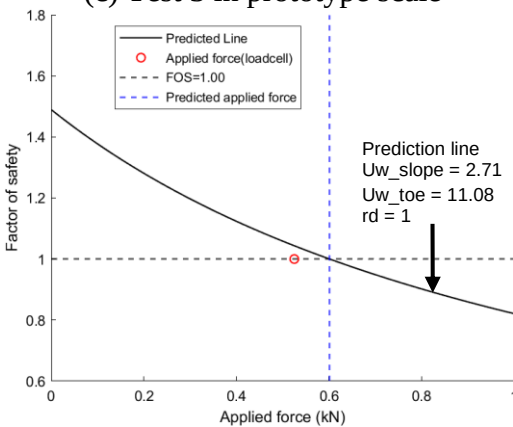
(b) Test 2 in prototype scale



(c) Test 3 in prototype scale



(d) Test 4 in prototype scale



(e) Test 5 in prototype scale

Figure 6-33 Prediction of applied force

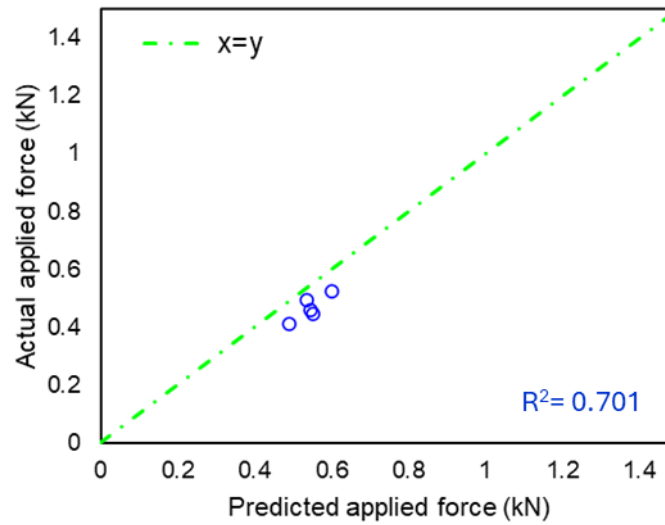


Figure 6-34 Correlation between predicted applied force and actual applied force

Figure 6-34 shows the comparison between the predicted and experimental applied forces. The coefficient of determination (R^2) is 0.701, suggesting that the predictions are reasonably estimate the applied force and surcharge weight needed to trigger failure in the dip slope model with counterweight under groundwater seepage conditions. However, to further validate the theoretical calculations, a variety of slope prototypes need to be investigated, as the applied force will vary with the scale of the prototype.

6.8 Summary

The primary focus of this chapter lies in the exploration of counterweight systems as a means of landslide prevention. Counterweight is constructed at toe part to prevent slope sliding. Chapter 6 delves into the investigation of dip slopes with various counterweight configurations, employing centrifuge models for experimental analysis.

- The research focused on a slope angle of 30 degrees with a toe angle of 0 degrees, aiming to compare two types of counterweights designed to exert an equivalent force on the toe bedding. Experimental findings, complemented by Particle Image Velocimetry (PIV) analysis, revealed that both counterweight types slide when subjected to water supply and surcharge atop the slope, resulting in a 1 cm displacement. However, the movement of both counterweight types ceased when the surcharge exceeded a displacement of 2 cm, while water continued to be supplied. Nevertheless, fore-thrust failure occurred near the top slope. These results suggest the counterweights' capability to alter the mode of failure from toe sliding to thrust failure.
- For another dip slope geometry with a slope angle of 30 degrees and a toe angle of 15 degrees, three different counterweight heights—1 cm, 1.75 cm, and 2 cm—were tested. The experimental results and PIV analysis indicated that toe sliding occurred with a counterweight height of 1 cm. However, fore-thrust failure was observed in the cases with counterweight heights of 1.75 cm and 2 cm. Thus, it can be concluded that a counterweight height of 1.75 cm is nearly optimal.
- The optimum height of the counterweight from theoretical calculations is 1.5 cm, which is consistent with the experimental results where 1.75 cm and 2 cm heights of the counterweight resulted in thrust failure.

Chapter 7 Centrifugal Model of Dip Slope on Permeable Bedding Plane with Groundwater Seepage and Passive Thrust Induced Instability

In this chapter, a series of centrifuge tests were conducted to investigate the failure mechanism of dip slopes on permeable bedding planes. Additionally, the counterweight fill described in Chapter 6 was adapted to dip slopes on permeable bedding planes. The slope geometry with a slope angle of 30 degrees and a reverse toe angle of 15 degrees was exclusively used in this chapter.

The experimental setup and methodology are explained in Sections 7.1 and 7.2, respectively. In Section 7.3, experimental results are reported. Considering the importance of the reduction ratio for slope stability calculations, the reduction ratio was calculated and reported in Section 7.4. In Section 7.5, the factor of safety was calculated and compared based on predictions and experimental results. Additionally, back-calculation methods were used to enhance the accuracy of the calculations by comparing them with experimental results. Finally, the findings from this chapter are summarized in Section 7.6.

7.1 Experimental setup and preparation of centrifuge physical model

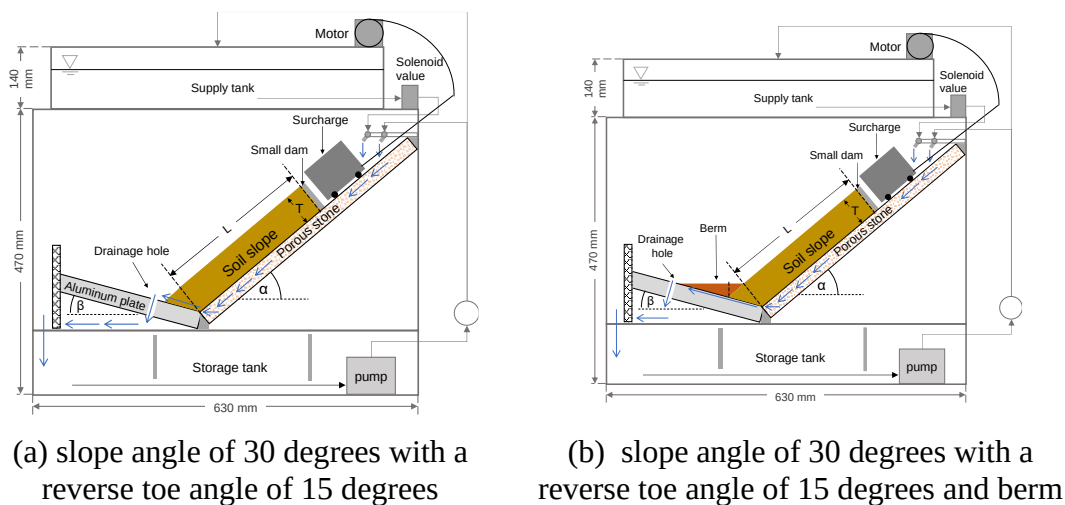


Figure 7-1 Schematic diagram of triggering for centrifuge model

The experiments comprised six cases, classified into two series. The first series entailed a slope angle of 30 degrees with a reverse toe angle of 15 degrees (refer to Figure 7-1(a)), while the second series involved a slope angle of 30 degrees with a reverse toe angle of 15 degrees and berm (refer to Figure 7-1(b)). During the experimental procedure, slope movement was induced by two factors: the surcharge weight, acting as a moving boundary to exert pressure on the slope in the model test, and watering, simulating groundwater seepage. As depicted in the schematic diagram in Figure 7-1, the surcharge load cart was controlled by a motor, while water supplementation was regulated by a solenoid valve connected to the top supply tank. In this chapter, a porous stone, acting as a permeable layer with a permeability coefficient of 5×10^{-5} m/s, is utilized. This high permeability allows water to easily penetrate the porous stone, causing water seepage to accumulate at the toe part of the slope.

The experimental material utilized in this study was Hiroshima sand, sourced from Kure Shi, Hiroshima prefecture. Initial processing involved sieving the sand to a diameter of 1mm. Subsequently, the sand was adjusted to a water content of 8% before compaction. Compaction was conducted to achieve a compaction degree of 90%. The basic properties of the Hiroshima sand are summarized in Chapter 3.

To determine the interface shear strength, tilting tests were conducted. Given the potential for soil sliding along the bedding plane, as well as the presence of lateral and toe supports, Teflon sheets were employed as the bedding plane, while aluminum materials were utilized for the lateral and toe supports in the experimental setup.

Table 7-1 Configuration of slope model lying bedding plane with lateral, toe supports and berm.

Test no.	Slope angle α (°)	Toe angle β (°)	Thickness (cm)	Width (cm)	Length (cm)	Berm Height (cm)	n_g
1	30	15	3	40	20	0	20
2	30	15	3	40	20	0	50
3	30	15	3	40	20	1	20
4	30	15	3	40	20	2	20
5	30	15	3	40	20	3	20
6	30	15	3	40	20	4	20

7.2 Method

Once the soil model was installed in the centrifuge room and connected to the data logger, centrifuge acceleration was increased to target G and maintained. The slope model was then triggered to fail using a combination of surcharge and water. However, the sequence of adding surcharge and water could vary. If water was added first without

triggering slope failure, surcharge was subsequently added, and the load cell recorded the applied force on the slope model during surcharge release. Conversely, if surcharge was added first and slope failure did not occur, water was then added to induce slope failure. If slope failure occurred after adding surcharge first, the experiment was conducted under dry conditions, with surcharge triggering the failure. During slope failure, high-speed cameras were employed to record the movement of the slope from a lateral perspective. Experimental procedures are given following:

Six experiments were conducted under the following conditions:

Test 1: 20G, watering1, watering2, surcharge

Test 2: 50G, watering, surcharge

Test 3: 20G, watering1, watering2, surcharge

Test 4: 20G, watering1, watering2, surcharge

Test 5: 20G, watering1, surcharge1, watering2, surcharge2

Test 6: 20G, watering1, surcharge1, watering2, surcharge2

7.3 Experimental result

After completing the experiment, the data collected from various sensors including laser sensors (measuring displacement in mm), load cell (recording drag force in N), earth pressure sensors (measuring in kPa), and pore water pressure sensors (measuring in kPa) were analyzed. The elapsed time was recorded and displayed during slope failure. The experimental results are comprehensively elucidated through the detailed analysis of Tests 1 to 6.

7.3.1 Test 1: Geometry condition $\alpha=30^\circ$, $\beta=15^\circ$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 20G, watering1, watering2, surcharge.

Figure 7-2 illustrates the response of instruments during the experimental procedure.

Drag force: Once the centrifugal acceleration reached 20G, the load cell measured a drag force of approximately 337.4N. Upon applying the surcharge to the slope, the drag force immediately increased to -31.82N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge.

Earth pressure: During water supplementation, the earth pressure remained constant. However, after the surcharge was applied to the slope, all earth pressures increased, with EP13 and EP14 showing a particularly notable increase.

Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, located near the toe slope, recorded a maximum pore pressure of 5.78 kPa. Then, water supply increased, PPT1, PPT2, PPT3 and PPT4, indicating water seepage accumulated at toe part of slope.

Accelerometer: Unfortunately, displacement laser sensors cannot be installed into the soil chamber. Therefore, three accelerometers were embedded in the slope to observe slope movement due to acceleration. The experimental results show that ACC1, ACC2, and ACC3 readings changed as the surcharge was applied to the slope.

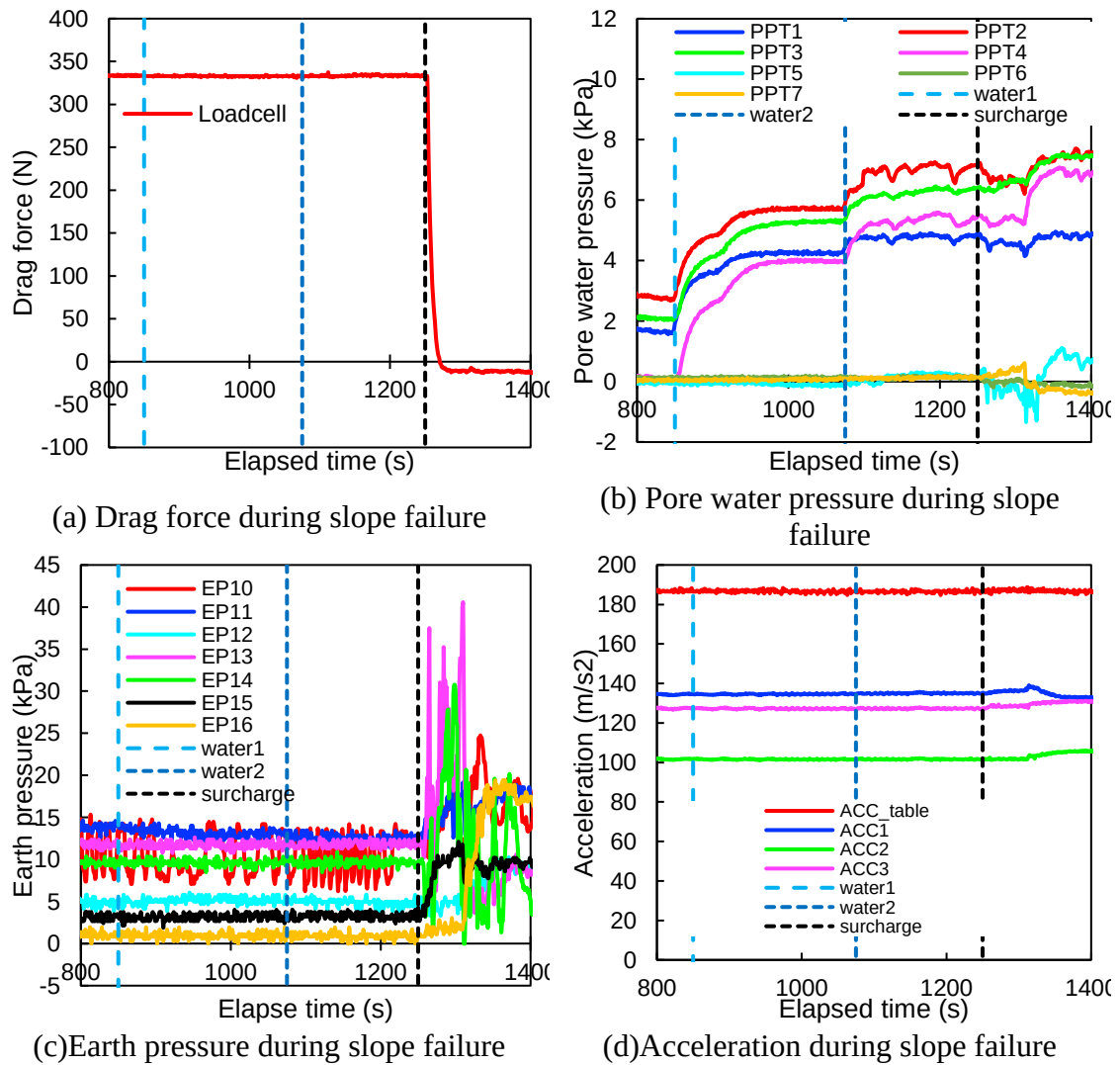


Figure 7-2 Experimental result of test 1

In Figure 7-3, the slope failure following the completion of the centrifuge experiment is depicted. The movement predominantly occurred at the toe slope, and upheaval was observed, as shown in Figure 7-3(b), indicating toe sliding. Additionally, during the

loading of the surcharge cart onto the slope, the movement of the surcharge cart was symmetrical, as depicted in Figure 7-3(a).

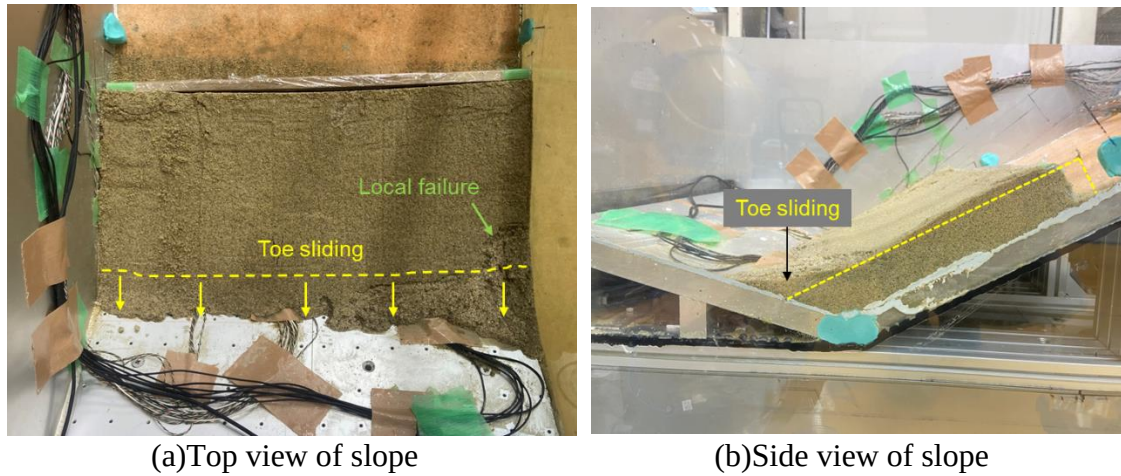


Figure 7-3 Slope failure observed after the testing of Test1

7.3.2 Test 2: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$

Test procedure: 50G, watering, surcharge.

Displacement: Upon reaching a centrifugal acceleration of 50G, there was observed movement in the displacement of the surcharge and horizontally. However, during water supplementation, the displacement from laser sensors remained stable. Subsequently, upon applying the surcharge, there was a significant increase in vertical and minimal horizontal displacement, indicating toe sliding

Drag force: Once the centrifugal acceleration reached 20G, the load cell measured a drag force of approximately -995.2N. Upon applying the surcharge to the slope, the drag force immediately increased to -241.1N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge.

Earth pressure: During water supplementation, the earth pressure remained constant. However, after the surcharge was applied to the slope, all earth pressures increased.

Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, located near the toe slope, recorded a maximum pore pressure of 10.96kPa. Additionally, seepage on slope is minimal, indicating water accumulated at toe. However, after the surcharge was applied, all pore pressures dropped.

In Figure 7-5, the slope failure following the completion of the centrifuge experiment is depicted. The movement predominantly occurred at the toe slope, and upheaval was observed on both slope and toe, as shown in Figure 7-5(b), indicating toe sliding.

Additionally, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was symmetrical, as depicted in Fig. 7-5(a).

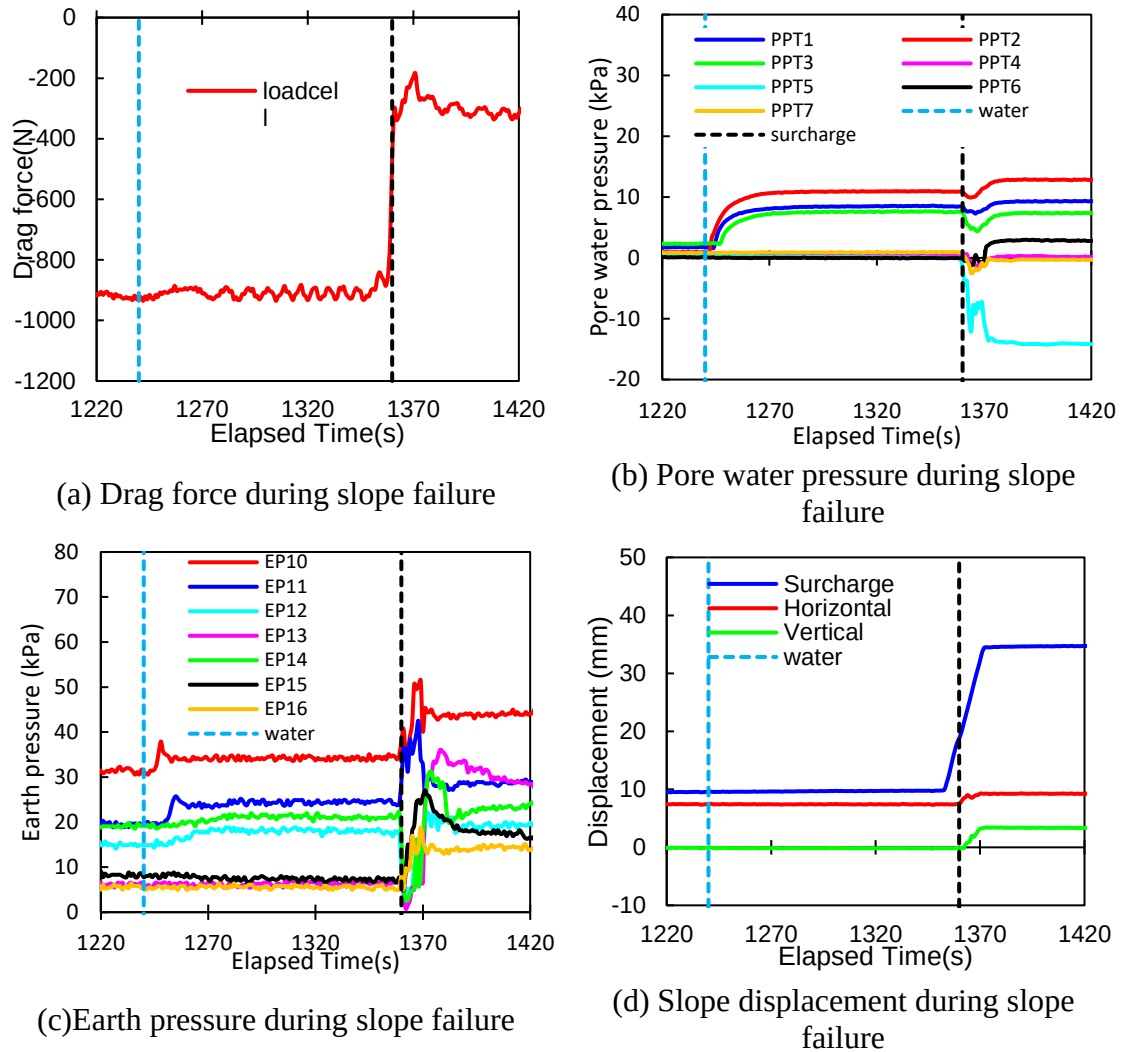


Figure 7-4 Experimental result of test 2

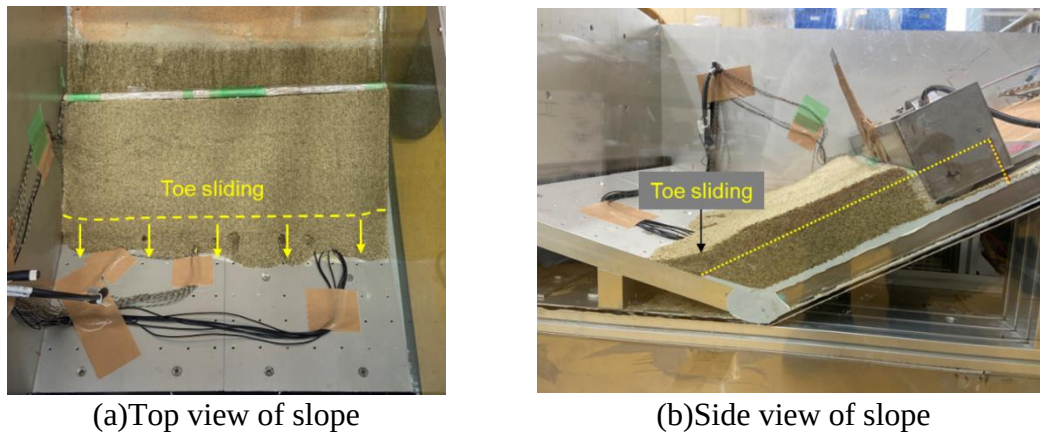


Figure 7-5 Slope failure observed after the testing of Test2

7.3.3 Test 3: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 1cm of height

Test procedure: 20G, watering1, watering2, surcharge

Drag force: Once the centrifugal acceleration reached 20G, the load cell measured a drag force of approximately 298.96N. Upon applying the surcharge to the slope, the drag force immediately decreased to -0.74N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge.

Earth pressure: During water supplementation, the earth pressure remained constant. However, after the surcharge was applied to the slope, all earth pressures increased slightly, with EP10 exhibiting fluctuations.

Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, positioned near the toe slope, recorded a maximum pore pressure of 5.78 kPa. Additionally, during water supplementation, PPT1, PPT2, PPT3, and PPT4 showed significant increases, indicating water seepage accumulated at the toe part of the slope. However, PPT5 increased further during the addition of more water flux.

Accelerometer: Unfortunately, displacement laser sensors cannot be installed into the soil chamber. Therefore, three accelerometers were embedded in the slope to observe slope movement due to acceleration. The experimental results show that ACC1, ACC2, and ACC3 readings changed as the surcharge was applied to the slope. ACC1 had significantly changed after surcharge applied to the slope, indicating toe sliding.

In Figure 7-7, the slope failure following the completion of the centrifuge experiment is depicted. The movement predominantly occurred at the toe slope, and upheaval was observed on both the slope and toe, as shown in Figure 7-7(b), indicating toe sliding. Additionally, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was symmetrical, as depicted in Figure 7-7(a). It should be noted that local and water erosion occurred near the glass wall side.

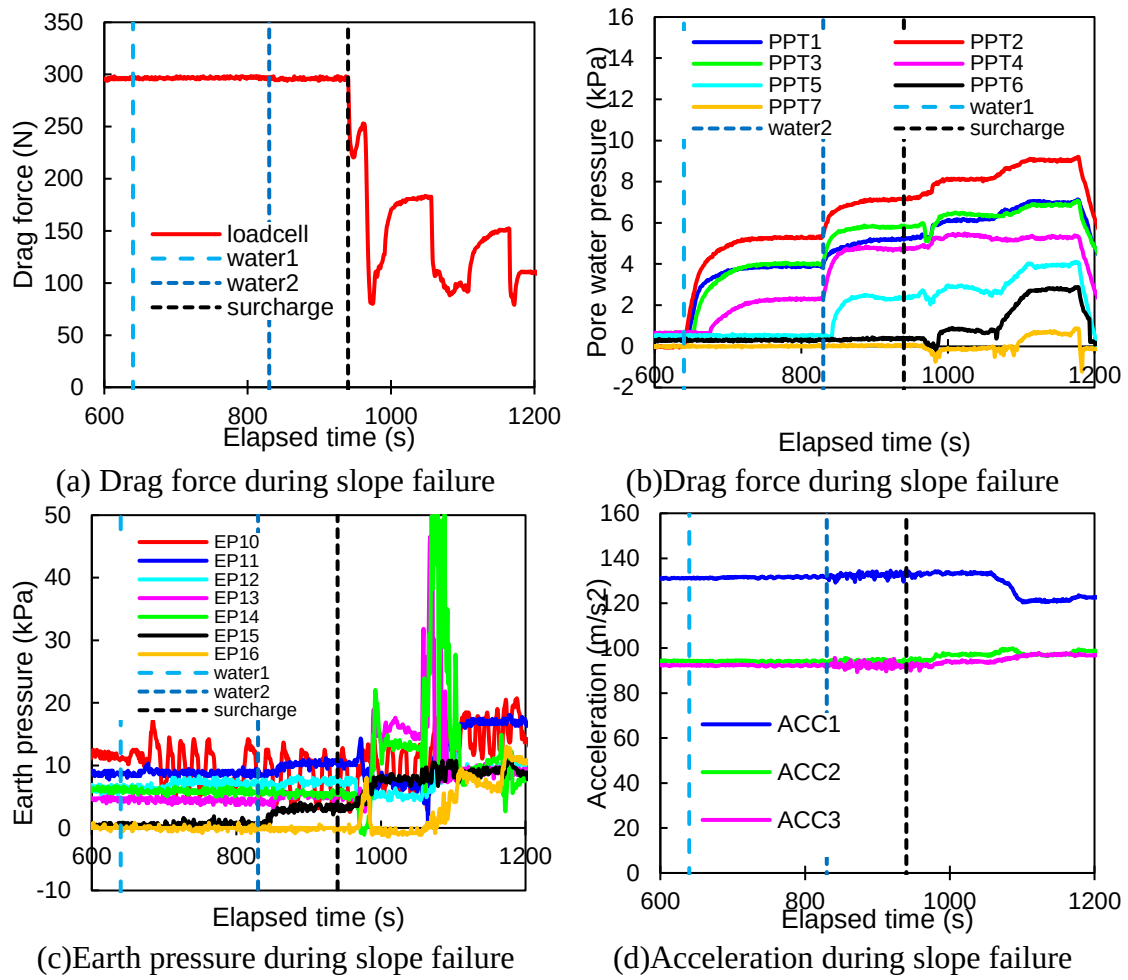


Figure 7-6 Experimental result of test 3

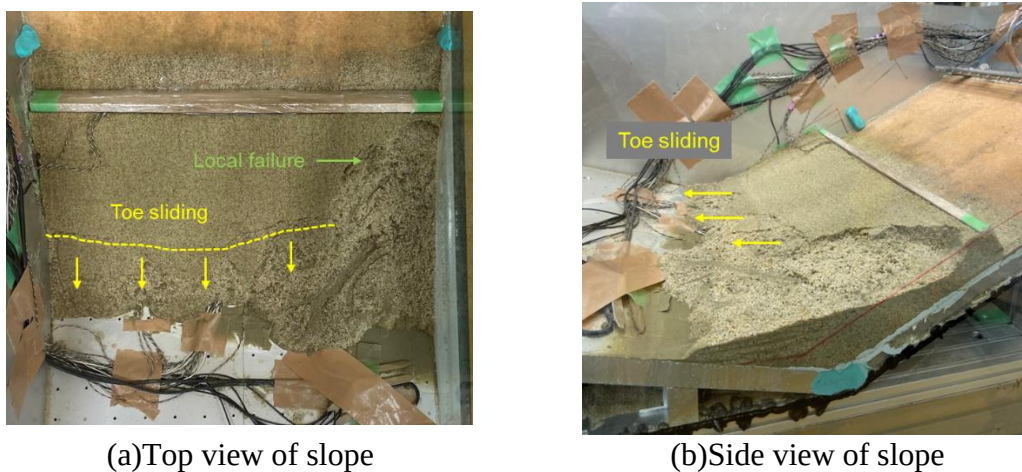


Figure 7-7 Slope failure observed after the testing of Test3

7.3.4 Test 4: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 2 cm of height

Test procedure: 20G, watering1, watering2, surcharge.

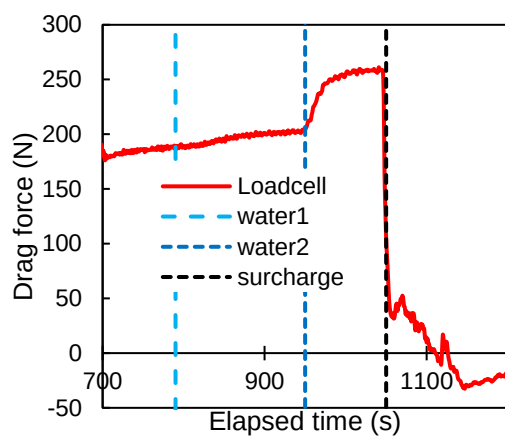
Drag force: Once the centrifugal acceleration reached 20G, the load cell measured a drag force of approximately 261.22N. Upon applying the surcharge to the slope, the drag force immediately decreased to -36.26N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge.

Earth pressure: During water supplementation, the earth pressure remained constant. However, after the surcharge was applied to the slope, all earth pressures increased immediately, with EP13 and EP14 showing a particularly notable increase.

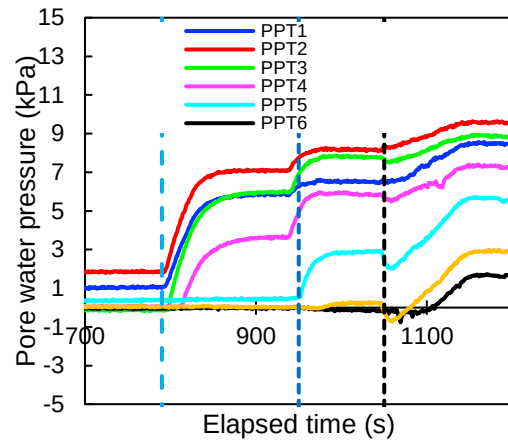
Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, positioned near the toe slope, recorded a maximum pore pressure of 8.23 kPa. Additionally, during water supplementation, PPT1, PPT2, PPT3, and PPT4 showed significant increases, indicating water seepage accumulated at the toe part of the slope. However, PPT5 increased further during the addition of more water flux.

Accelerometer: Unfortunately, displacement laser sensors cannot be installed into the soil chamber. Therefore, three accelerometers were embedded in the slope to observe slope movement due to acceleration. The experimental results show that ACC1, ACC2, and ACC3 readings changed as the surcharge was applied to the slope. ACC1 had significantly changed after surcharge applied to the slope, indicating toe sliding.

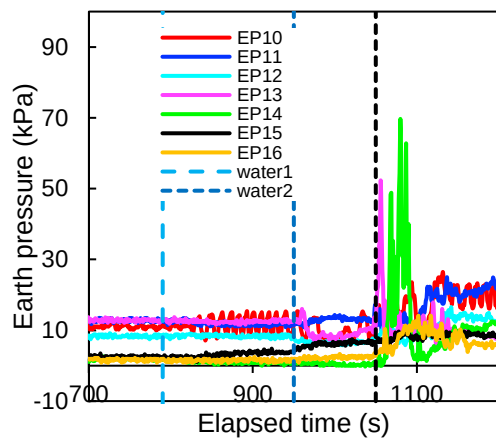
In Figure 7-9, the slope failure following the completion of the centrifuge experiment is depicted. The movement predominantly occurred at the toe slope, and upheaval was observed on both the slope and toe, as shown in Figure 7-9(b), indicating toe sliding and thrust failure. Additionally, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was symmetrical, as depicted in Figure 7-9(a). It should be noted that local and water erosion occurred near the glass wall side.



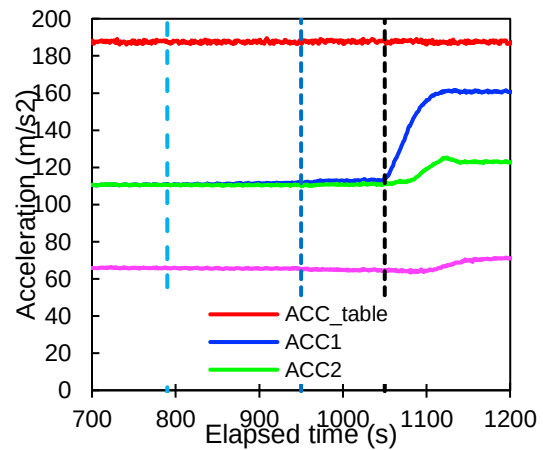
(a) Drag force during slope failure



(b) Pore-water pressure during slope failure

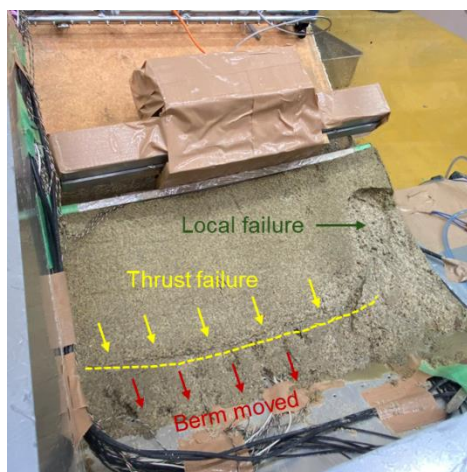


(c) Earth pressure during slope failure



(d) Acceleration during slope failure

Figure 7-8 Experimental result of test 4



(a) Top view of slope



(b) Side view of slope

Figure 7-9 Slope failure observed after the testing of Test4

7.3.5 Test 5: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 3 cm of height

Test procedure: 20G, watering1, surcharge1, watering2, surcharge2

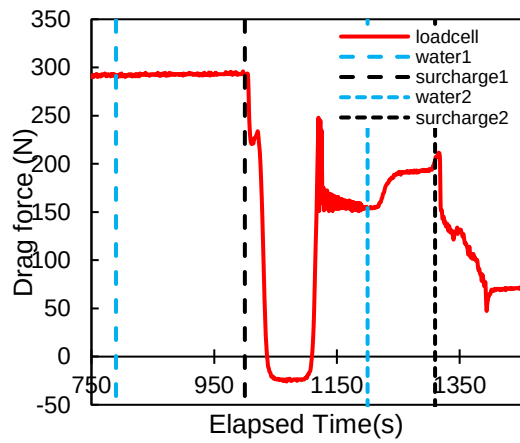
Drag force: Once the centrifugal acceleration reached 20G, the load cell measured a drag force of approximately 296N. Upon applying the surcharge to the slope, the drag force immediately decreased to -25.9N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge.

Earth pressure: During water supplementation, the earth pressure remained constant. However, after the surcharge was applied to the slope, all earth pressures increased immediately, with EP10 and EP11 showing a particularly notable increase.

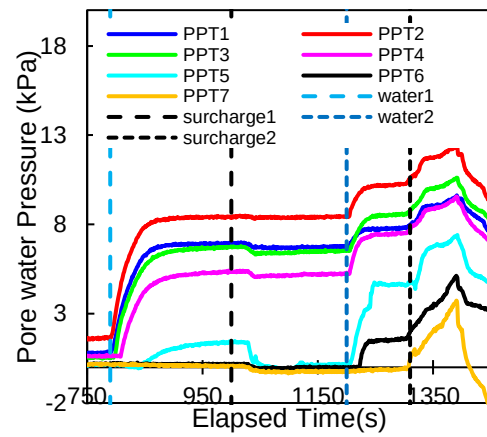
Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, positioned near the toe slope, recorded a maximum pore pressure of 8.23 kPa. Additionally, during water supplementation, PPT1, PPT2, PPT3, and PPT4 showed significant increases, indicating water seepage accumulated at the toe part of the slope. However, PPT5 increased further during the addition of more water flux.

Accelerometer: Unfortunately, displacement laser sensors cannot be installed into the soil chamber. Therefore, three accelerometers were embedded in the slope to observe slope movement due to acceleration. The experimental results show that ACC1, ACC2, and ACC3 readings changed as the surcharge was applied to the slope. ACC2 and ACC3 had significantly changed after surcharge applied to the slope, indicating thrust failure.

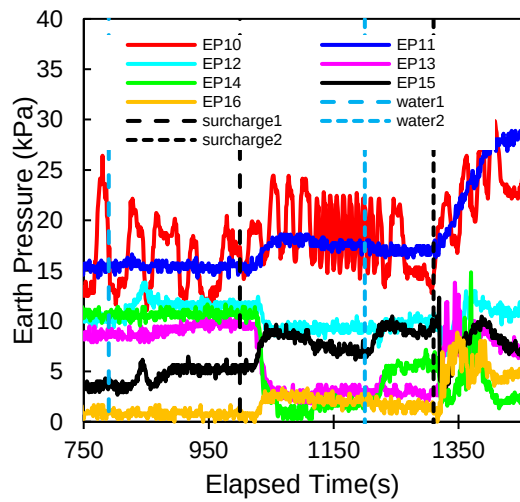
In Figure 7-11, the slope failure following the completion of the centrifuge experiment is illustrated. The movement predominantly occurred at the slope part near berm, and upheaval was observed, as shown in Figure 7-11(b), indicating toe sliding and thrust failure. Additionally, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was unsymmetrical, as depicted in Figure 7-11(a). It should be noted that local and water erosion occurred near the glass wall side.



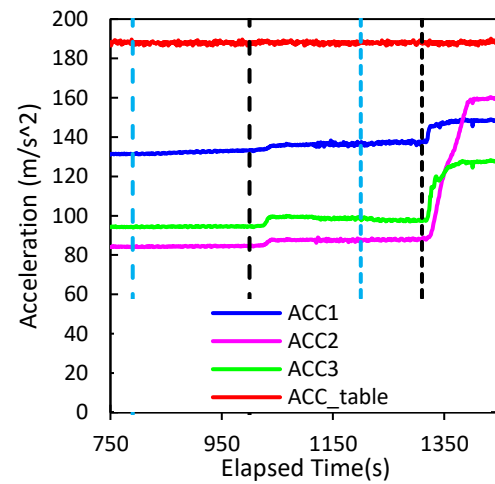
(a) Drag force during slope failure



(b) Pore-water pressure during slope failure



(c) Earth pressure during slope failure



(d) Acceleration pressure during slope failure

Figure 7-10 Experimental result of test 5



(a) Top view of slope



(b) Side view of slope

Figure 7-11 Slope failure observed after the testing of Test5

7.3.6 Test 6: Geometry condition $\alpha=30$, $\beta=15$, $T = 3\text{cm}$, $B = 40\text{cm}$, $L = 20\text{cm}$ and Berm: 4 cm of height

Test procedure: 20G, watering1, surcharge1, watering2, surcharge2.

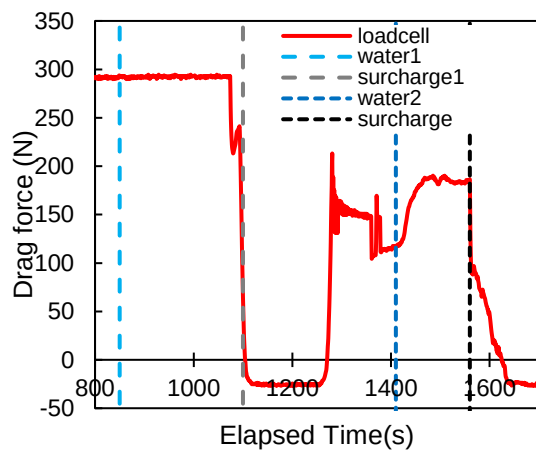
Drag force: Once the centrifugal acceleration reached 20G, the load cell measured a drag force of approximately 294.52N. Upon applying the surcharge to the slope, the drag force immediately decreased to -28.12N. This increase in drag force was attributed to the surcharge making contact with the slope and exerting a force, which differed from the drag force observed before and after the addition of the surcharge.

Earth pressure: During water supplementation, the earth pressure remained constant. However, after the surcharge was applied to the slope, all earth pressures increased immediately, with EP10, EP11 and EP16 showing a particularly notable increase.

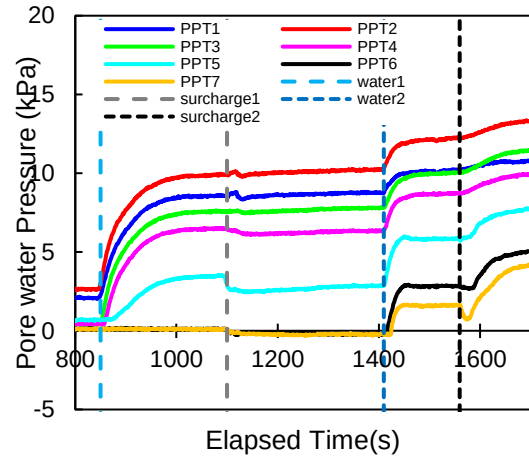
Pore-water pressure: During water supplementation, there was a significant increase in pore water pressure. PPT2, positioned near the toe slope, recorded a maximum pore pressure of 8.23 kPa. Additionally, during water supplementation, PPT1, PPT2, PPT3, and PPT4 showed significant increases, indicating water seepage accumulated at the toe part of the slope. However, PPT5 increased further during the addition of more water flux.

Accelerometer: Unfortunately, displacement laser sensors cannot be installed into the soil chamber. Therefore, three accelerometers were embedded in the slope to observe slope movement due to acceleration. The experimental results show that ACC1, ACC2, and ACC3 readings changed as the surcharge was applied to the slope. ACC2 had significantly changed after surcharge applied to the slope, indicating thrust failure near berm.

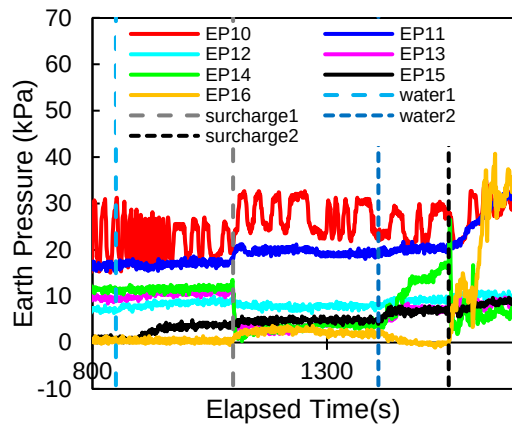
In Figure 7-13, the slope failure following the completion of the centrifuge experiment is illustrated. The movement predominantly occurred at the slope part near berm, and upheaval was observed, as shown in Figure 7-13(b), indicating toe sliding and thrust failure. Additionally, during the loading of the surcharge cart onto the slope, the movement of the surcharge cart was unsymmetrical, as depicted in Figure 7-13(a).



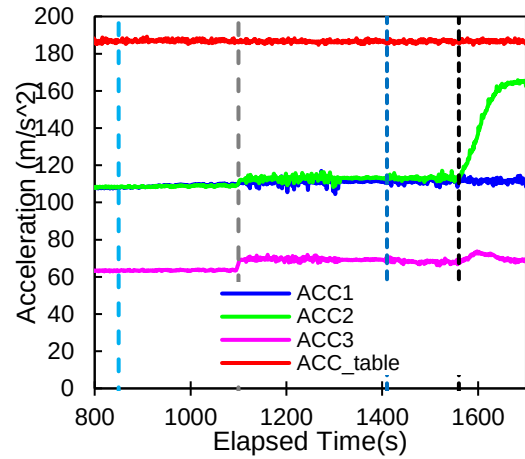
(a) Drag force during slope failure



(b) Pore water pressure during slope failure

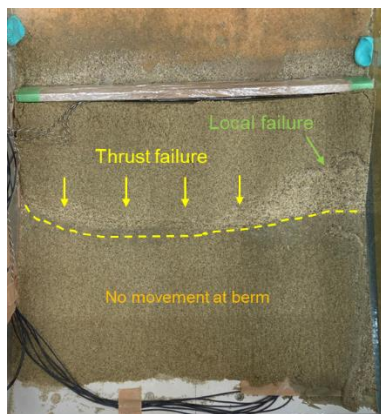


(c) Earth pressure during slope failure

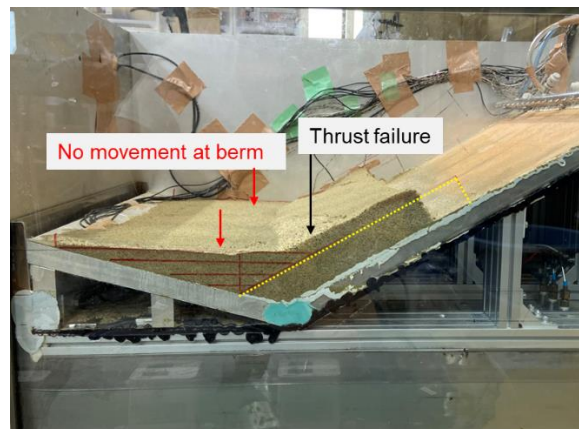


(d) Acceleration during slope failure

Figure 7-12 Experimental result of test 5



(a) Top view of slope



(b) Side view of slope

Figure 7-13 Slope failure observed after the testing of Test5

7.4 Reduction ratio

The experimental reduction ratio (r_d) is depicted in Figure 7-14. In Test 1, shown in Figure 7-15(a), r_{d1} and r_{d3} remained relatively unchanged during water supplementation, but both increased significantly upon the application of the surcharge. This value is crucial for subsequent calculations. Test 2, as shown in Figure 7-15(b), exhibited minimal change in r_{d1} and r_{d2} during water supplementation, but r_{d1} experienced a substantial decrease upon applying the surcharge to the slope. In Test 3 (Figure 7-15(c)), all r_d values increased immediately upon adding the surcharge, with r_{d1} showing a notable increase. Figure 7-15(d) illustrates r_d in Test 4, where r_d slightly increased during water supplementation, but significantly decreased upon adding the surcharge to the slope. In Test 5, all r_d values increased immediately after adding the surcharge, with no change during water supply, as shown in Figure 7-15(e). Finally, in Test 6, r_d remained unchanged during water supplementation but jumped upon applying the surcharge to the slope, as depicted in Figure 7-15(f).

The experimental results suggest a rise in the reduction ratio, reaching a peak around 1, which corresponds closely with the calculation of passive earth pressure. These experimental reduction ratios will be employed in theoretical calculations.

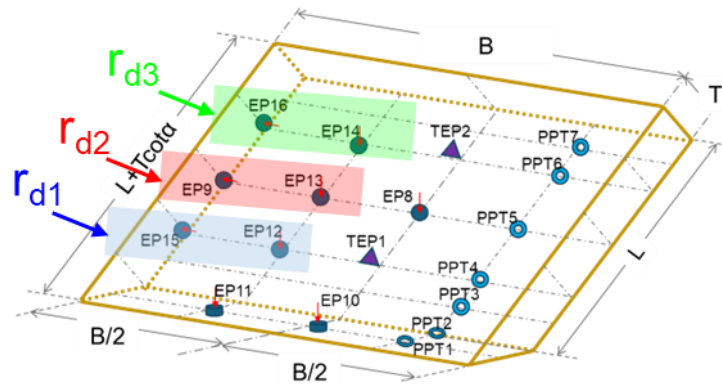


Figure 7-14 location of earth pressure gauge and pore water pressure sensors for calculating reduction ratio

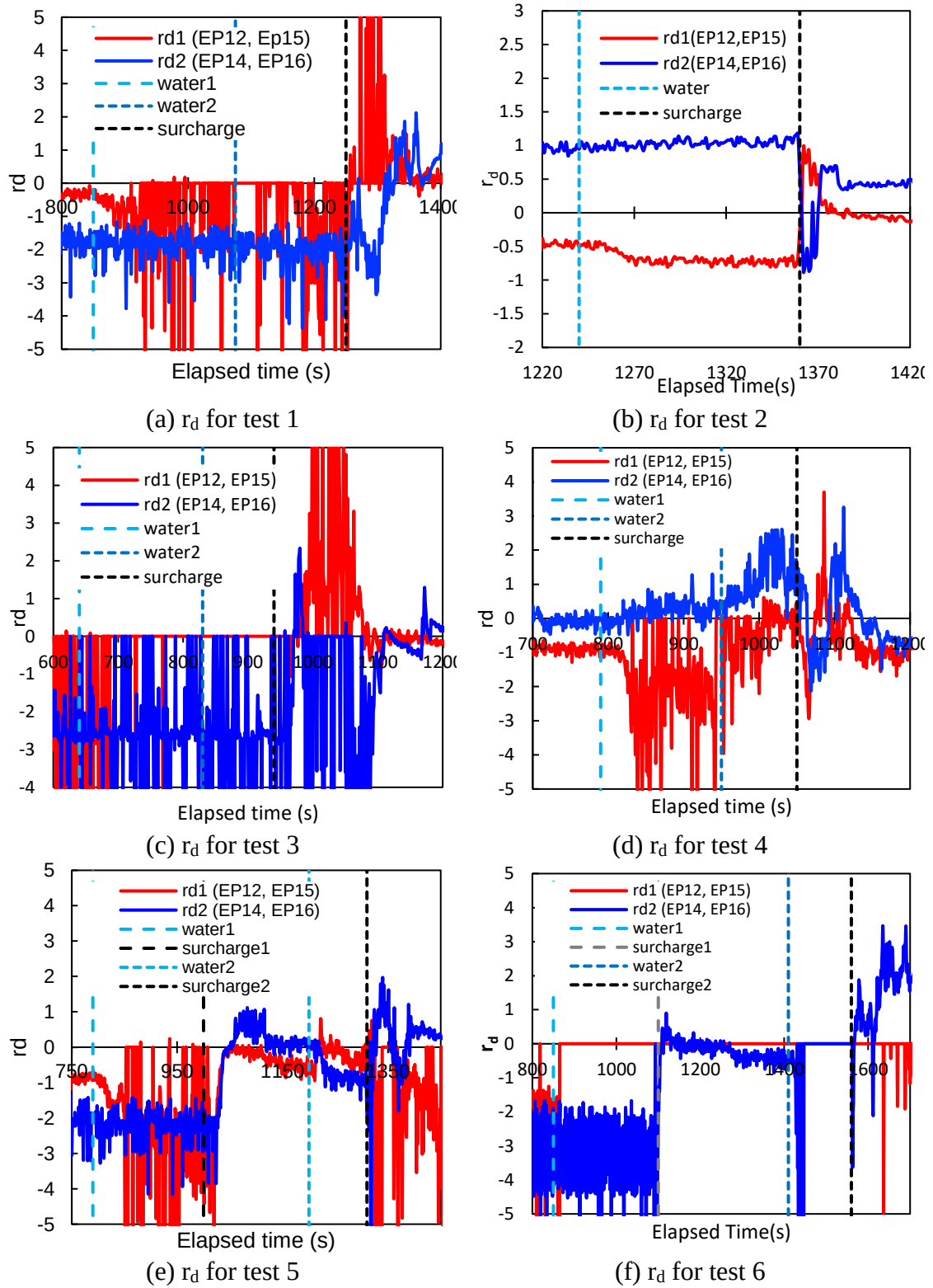


Figure 7-15 Experimental reduction ratio

7.5 Discussion

Chapter 5 and Chapter 6 examined parallel seepage along the bedding plane. However, in this chapter, groundwater accumulates at the toe due to the permeable bedding. The groundwater profile is adjusted to match the experimental groundwater conditions. Figure 7-16 shows the slope divided into three parts: (1) near the crest, (2) from the toe to the middle, and (3) the toe part.

In part (1), there is no groundwater as the water flows into the porous stone. In part (2), groundwater accumulates in the middle of the slope. In part (3), water fully saturates the toe. Therefore, pore-water pressure from the experimental results will be analyzed and applied to the theoretical calculations in Section 5.1 and Section 6.1.

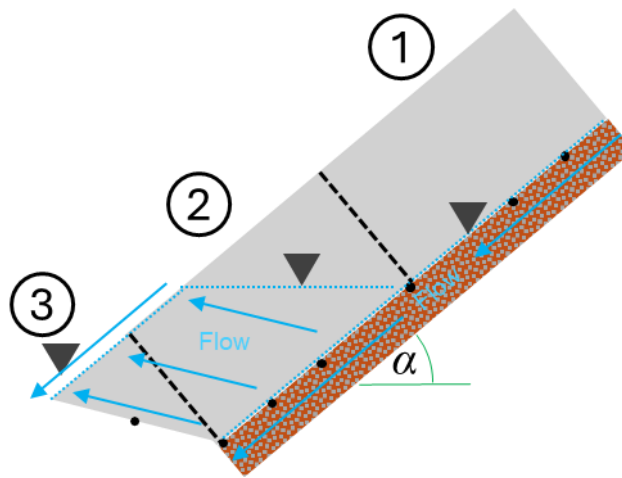
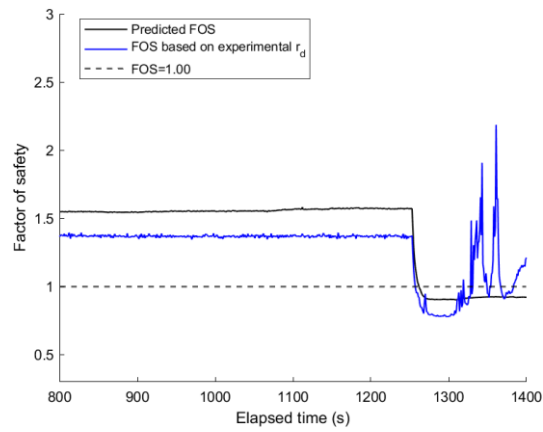


Figure 7-16 Adjusted groundwater of dip slope on permeable bedding plane

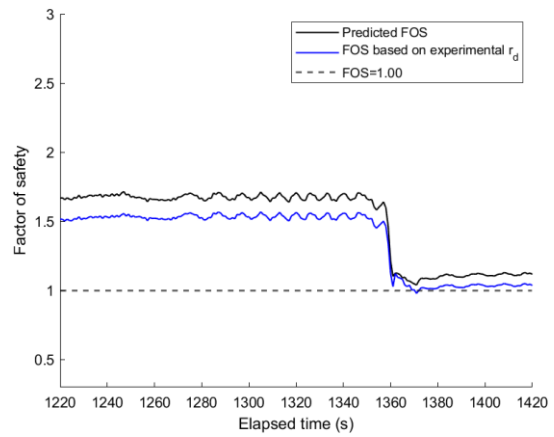
7.5.1 Factor of safety (FOS)

The Factor of Safety (FOS) can be calculated based on the resistance force and driving force. In the experiment, pore-water pressure transducers and applied force measurements were recorded. These results can be analyzed in terms of FOS, as illustrated in Figure 7-17. To maximize lateral resistance and predict FOS, a reduction ratio (r_d) of +1 (passive) was assumed. Subsequently, the experimental r_d values were used for calculation and comparison with predictions.

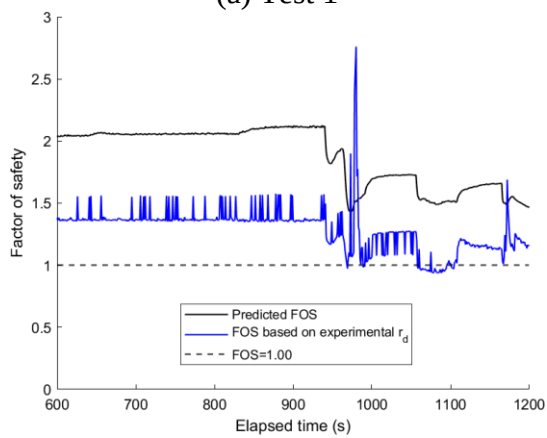
In test 1, the experimental FOS closely matches the predictions. However, from test 2 to test 6, the predicted FOS is not less than 1.0, indicating that the slope did not fail.



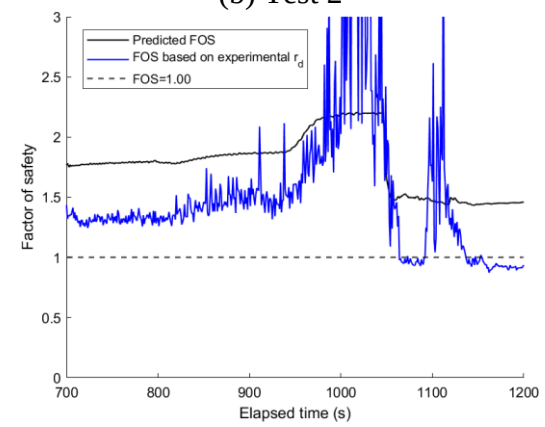
(a) Test 1



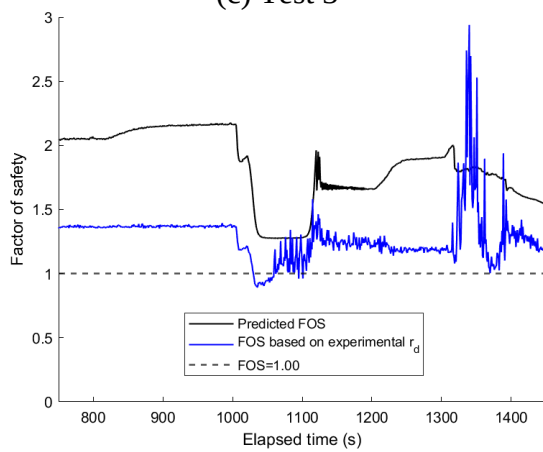
(b) Test 2



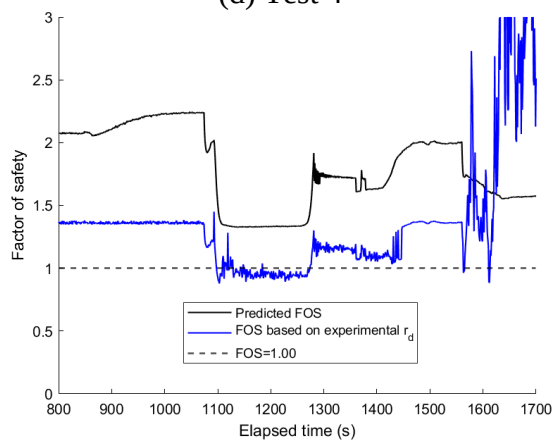
(c) Test 3



(d) Test 4



(e) Test 5



(f) Test 6

Figure 7-17 Comparison between prediction and experimental factor of safety

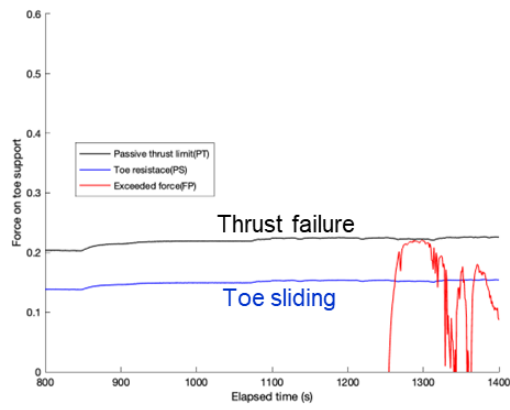
7.5.2 Failure mode and optimum counterweight

In Section 5.1, theoretical calculations are provided for failure mode analysis of slopes without counterweight fill in Eq. (5-20). In Section 6.1, theoretical calculations are provided for failure mode analysis of slopes with counterweight fill in Eq. (6-6). Toe resistance (PS) and passive thrust limitation (PT) can be used to determine the failure mode of the slope. Figure 7-18 illustrates the comparison between PS and PT; it can be seen that PS is lower than PT in tests 1 and 2 (slopes without counterweight), indicating toe sliding.

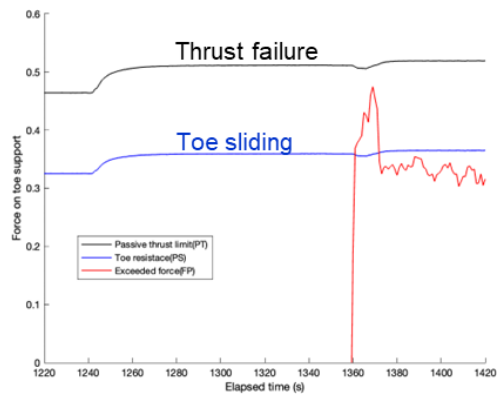
Moreover, the slope with a 1 cm height of counterweight in Figure 7-18(c) shows PS is less than PT, indicating toe sliding and that the counterweight fill was insufficient. However, tests 4, 5, and 6 depicted PS as higher than PT, indicating thrust failure. The calculations of the failure mode align with the experimental results presented in Section 7.3.

7.5.3 Back calculation of applied force

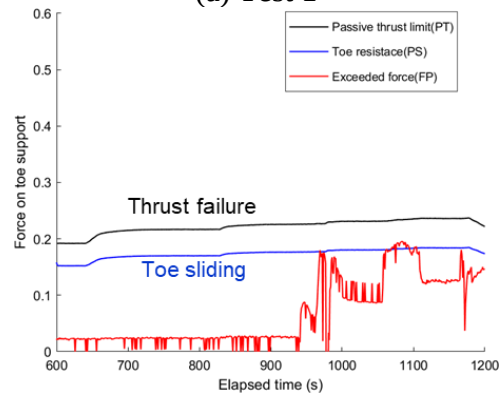
To enhance the accuracy of theoretical calculations, back-calculation of the applied force is of particular interest. Figure 7-19 illustrates the relationship between FOS and applied force. The prediction line represents slope stability considering existing pore-water pressure and $r_d = 1$. The applied force is increased until the FOS reaches 1.00, indicating slope failure. This predicted applied force is then compared with the applied force that led to slope failure in the experiments.



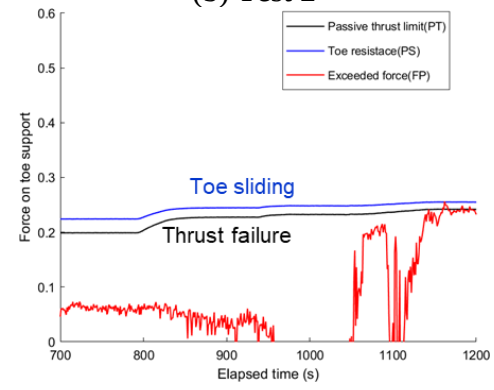
(a) Test 1



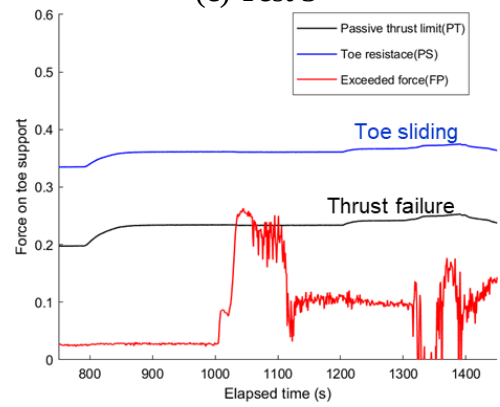
(b) Test 2



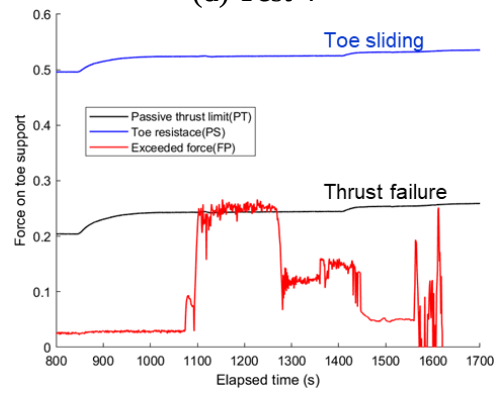
(c) Test 3



(d) Test 4

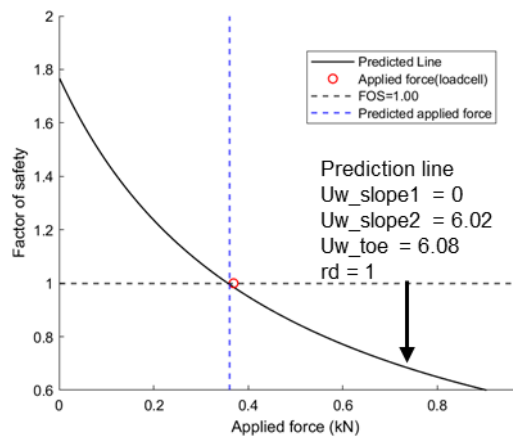


(e) Test 5

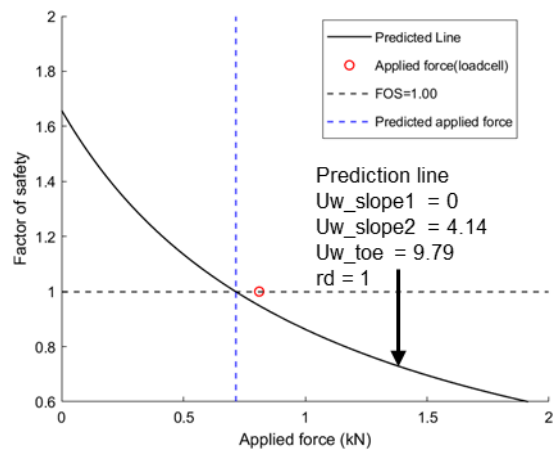


(f) Test 6

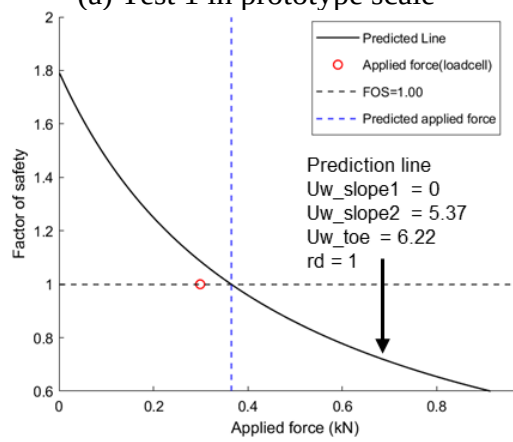
Figure 7-18 Comparison of toe resistance and passive thrust limitation



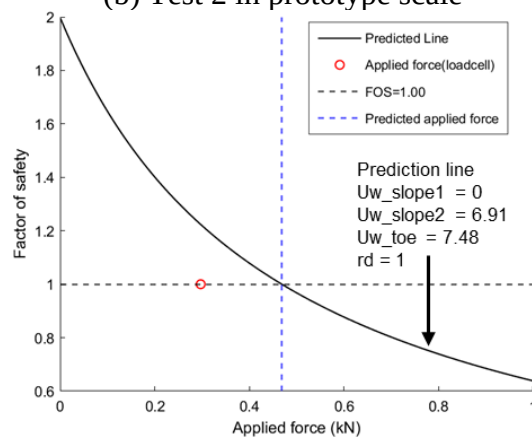
(a) Test 1 in prototype scale



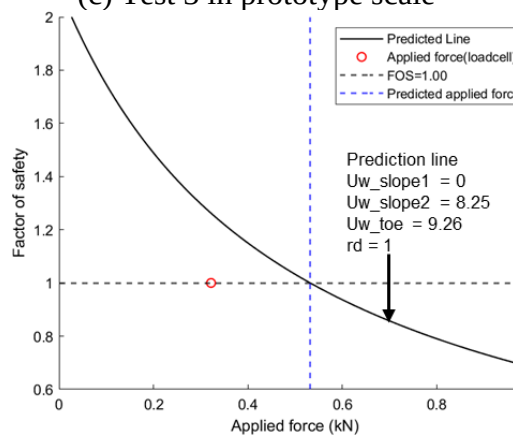
(b) Test 2 in prototype scale



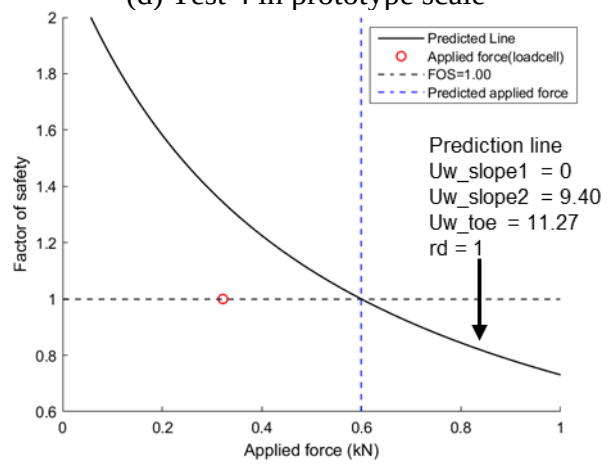
(c) Test 3 in prototype scale



(d) Test 4 in prototype scale



(e) Test 5 in prototype scale



(f) Test 6 in prototype scale

Figure 7-19 Prediction of applied force

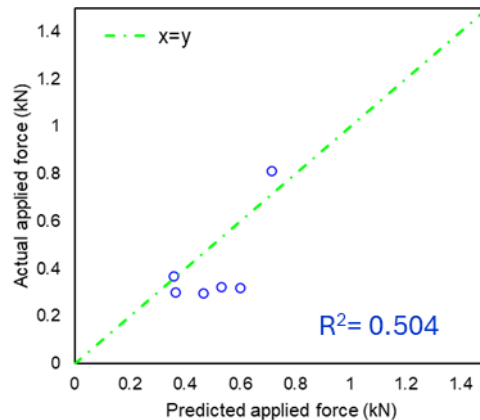


Figure 7-20 Correlation between predicted applied force and actual applied force

Figure 7-20 shows the comparison between the predicted and experimental applied forces. The coefficient of determination (R^2) is 0.504, indicating that the back-calculation for the applied force requires additional experimentation and that the theoretical calculations need to be adjusted.

7.6 Summary

In summary, dip slopes lying on permeable bedding planes and the effect of counterweight mitigation were investigated through centrifugal modeling. The slope geometry used had a slope angle of 30 degrees with a reverse toe angle of 15 degrees. The experimental results revealed that groundwater seepage accumulated at the toe of the slope. Moreover, the failure mode of slopes without counterweight was identified as toe sliding.

For slopes with counterweight fill, a counterweight height of 1 cm was insufficient to mitigate the dip slope, consistent with results in Chapter 6. However, slopes with counterweight fills ranging from 2 cm to 4 cm exhibited thrust failure. The failure modes observed in this chapter were similar to those in Chapter 5 and Chapter 6, but with different groundwater seepage patterns. Groundwater seepage was adjusted to match the experimental results, which showed accumulation at the toe (Section 7.5).

Experimental FOS indicated slope collapse when it was less than 1.0. However, the predicted FOS did not show collapse, as it never fell below 1.0. The discrepancy is due to the assumption of a r_d value of +1.0 in the theoretical predictions, which rendered the resistance force unreliable. This issue requires further investigation.

Chapter 8 Pseudo-static Stability Analysis of a Laterally Confined Dip Slope Investigated by Dynamic Centrifugal Model

In this chapter, one centrifuge test was conducted to investigate the failure mechanism of dip slopes subjected to seismic loading. The slope geometry with a slope angle of 40 degrees and a toe angle of 0 degrees was exclusively used.

Theoretical calculations of pseudo-static stability are provided in Section 8.1. The model description and experimental procedures are explained in Sections 8.2 and 8.3, respectively. In Section 8.4, experimental results are reported. In Section 8.5, the reduction ratio is discussed. Additionally, the factor of safety was calculated and compared based on predictions and experimental results. Finally, the findings obtained in this chapter are summarized in Section 8.6.

8.1 Theoretical calculations of slope lying on bedding plane with lateral and toe supports subjected to seismic load

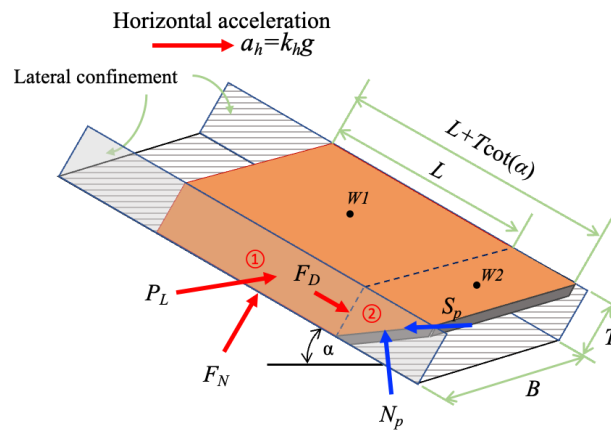


Figure 8-1 Laterally confined slope lying on bedding plane and toe support.

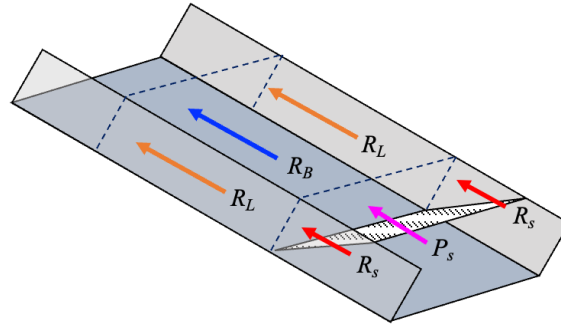


Figure 8-2 The resistance force against slope sliding.

This study employs the concept of dip slope with lateral confinements and toe support. It takes into account seismic effects through the application of the pseudo-static method. The configuration of the inclined surface is illustrated in Figure 8-1. Therefore, the dip slope can be divided into two distinct sections: ① the main slope block, and ② the toe segment. The gravitational force acting on the soil is represented by Eqs. (8-1) and (8-2). Furthermore, the constituent forces of the soil's weight (the normal force, F_N and the driving force, F_D) are determined by Eqs. (8-4) to (8-5).

$$W_1 = \gamma BTL \quad (8-1)$$

$$W_2 = \frac{\gamma BT^2}{2} \cot(\alpha) \quad (8-2)$$

$$F_D = W_1(\sin(\alpha) + k_h \cos(\alpha)) \quad (8-3)$$

$$F_{N1} = W_1(\cos(\alpha) - k_h \sin(\alpha)) \quad (8-4)$$

$$F_{N2} = W_2 \quad (8-5)$$

where

- α initial angle of slope model
- k_h horizontal seismic coefficient
- γ bulk unit weight of slope model
- B breadth of the contact area along the slope model
- T thickness of slope model
- L length of the contact area along the slope model

In the conceptual model with confinements, the forces opposing the slope's movement represent the resistance at the interface, encompassing the resistance on the bedding plane and the confinements provided laterally. Within the framework of the slope model, the shear resistance along the bedding plane (R_B) is governed by the contact area of the bedding. Additionally, the shear resistance laterally (R_L) is dictated by the contact areas of the lateral support. Consequently, the equations for determining R_B and R_L are provided in Eqs. (8-6) and (8-12) respectively. While F_{N1} is employed directly to compute R_B , the computation of R_L is more intricate due to the fact that the frictional force relies on the average pressure exerted on the laterally confined surface (P_L).

As outlined by Pipatpongsa et al. (2022), the mean pressure exerted on the

bedding plane can be straightforwardly defined using Eq. (8-7). However, lateral confinement is linked to the stress history of the slope model. Thus, the calculation of P_L involves a synthesis of the notion of reduction in shear strength and the reduced Rankine's passive earth pressure, represented by Eq. (8-8).

$$R_B = F_{N1} \tan(\phi_i) + c_i LB \quad (8-6)$$

$$P_{N1} = \frac{F_N 1}{BL} \quad (8-7)$$

$$P_{L1} = P_{N1} K_p + 2c_d \sqrt{K_p} \quad (8-8)$$

$$K_p = \tan\left(\frac{\pi}{4} + \frac{\phi_d}{2}\right)^2 \quad (8-9)$$

The shear strength parameters in terms of effective stress, c' , and $\tan(\phi')$, can be reduced before the slope reaches a state of limiting equilibrium (factor of safety =1.0) (Bishop and Morgenstern, 1960). Consequently, the factor of safety (FS) can define how much reduction in shear strength to reach failure and it is linked to the reduction factor (r_d), (Dawson, 1999; Matsui and San, 1992).

$$\tau = \frac{c'}{FS} + \sigma'_n \cdot \frac{\tan \phi'}{FS} \quad (8-10)$$

$$r_d = \frac{1}{FS} = \frac{\tan(\phi_d)}{\tan(\phi)} = \frac{c_d}{c} \quad (8-11)$$

$$\begin{aligned} R_L &= \int_0^T (P_{L1} \tan(\phi_s) + c_s) L dt \\ &= \left[\left(\frac{\gamma T K_p}{2} (\cos(\alpha) - k_h \sin(\alpha)) + 2c_d \sqrt{K_p} \right) \tan(\phi_s) + c_s \right] LT \end{aligned} \quad (8-12)$$

The range of r_d spans from -1 to 1. Negative values of r_d indicate slopes in an active state, r_d equaling 0 signifies isotropic conditions, and positive r_d values denote slopes in a passive state.

Furthermore, the force that remains (F_P) from the upper portion of the slope exerts pressure on the toe segment, following Eq. (8-16). The potential for toe sliding emerges when F_P surpasses the resistance of the toe segment. At the termination point of the slope, the toe support experiences the impact of three forces: the normal force (N_p), the shear force (S_p), and the lateral resistance (R_s). These three forces can be described by Eqs. (8-17), (8-18) and (8-15), respectively. Nonetheless, the onset of slope sliding occurs when the shear force, S_p , surpasses the resistance offered by the toe support. This condition is encapsulated in inequality (8-19). To elaborate, substituting Eqs. (8-17) and (8-18) into inequality (8-19) yields a comprehensive representation. Furthermore, the calculation of F_P , equated to the passive thrust resistance (P_s) becomes pivotal, as the sliding initiation at the toe commences when $F_P > P_s$.

$$P_{N2} = \frac{F_{N2}}{BT \cdot \csc(\alpha)} \quad (8-13)$$

$$P_{L2} = P_{N2}K_p + 2c_d\sqrt{K_p} \quad (8-14)$$

$$R_s = \int_0^T (P_{L2}\tan(\phi_s) + c_s)t \cdot \cot(\alpha)dt \quad (8-15)$$

$$= \left[\left(\frac{\gamma TK_p}{3} \cos(\alpha) + 2r_d c \sqrt{K_p} \right) \tan(\phi_s) + c_s \right] \frac{T^2 \cot(\alpha)}{2} \quad (8-16)$$

$$F_p = F_D - R_B - 2R_L \quad (8-17)$$

$$N_p = F_p \sin(\alpha) + W_2 \quad (8-18)$$

$$S_p = F_p \cos(\alpha) + W_2 \cdot k_h - 2R_s \quad (8-19)$$

$$S_p \leq N_p \tan(\phi_t) + c_t BT \cdot \csc(\alpha) \quad (8-20)$$

$$F_p = P_s = \frac{(c_t BT \cdot \csc(\alpha) + 2R_s) + W_2 \sin(\phi_t) - k_h \cos(\phi_t)}{\cos(\alpha + \phi_t)}$$

Therefore, total resistance (R_T) can be combined as Eq. (8-21)

$$R_T = R_B + 2R_L + P_s \quad (8-21)$$

Multiple approaches for analyzing slope stability encompass static equilibrium, probabilistic, finite difference, and elemental techniques. Among these, the limit equilibrium method is the most commonly employed and direct technique to evaluate the potential for slope failure, relying on both slope geometry and the conditions of soil. The factor of safety (FS) for the slope is computed as the division of the resistance force (R_T) by the driving force (F_D).

$$FS = \frac{R_T}{F_D} \quad (8-22)$$

8.2 Model description

A soil chamber made of aluminum alloy with a transparent reinforced glass designed for visual monitoring was used in this experiment. The chamber had dimensions of 63×20×30 cm³ (length × width × height). The experimental setup included a schematic representation of the slope model, shown in Figure 8-3. The aluminum frame was installed and covered by Teflon sheet to serve as a bedding and toe support for the dip slope model. The slope geometry comprised a 40° slope angle, a thickness of 50 mm, a width of 200 mm, and a length of 200 mm. The slope model was constructed using Edosaki sand, of which material and physical properties is shown in Table 8-1. The Edosaki sand was compacted with a controlled 90% degree of compaction and a moisture content of 8% for this experiment. Figure 8-5 depicts the dip slope model following its preparation. Measurement instruments were affixed to the Teflon sheet at the bedding, toe of the slope and the chamber wall (as illustrated in Figure 8-4). To record changes in earth pressure within the soil model, 7 earth pressure gauges were installed. Additionally,

3 accelerometers were placed in the soil model to monitor slope dynamic behavior. Displacements were tracked using 2 laser sensors, capturing both vertical and horizontal movements. Notably, this study employed a centrifugal acceleration of 20G.

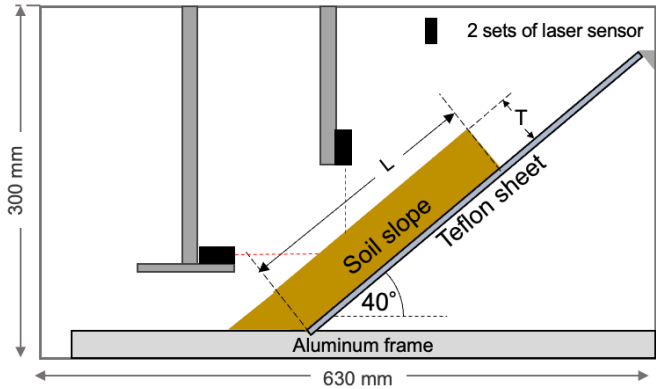


Figure 8-3 Schematic diagram of the experimental setup

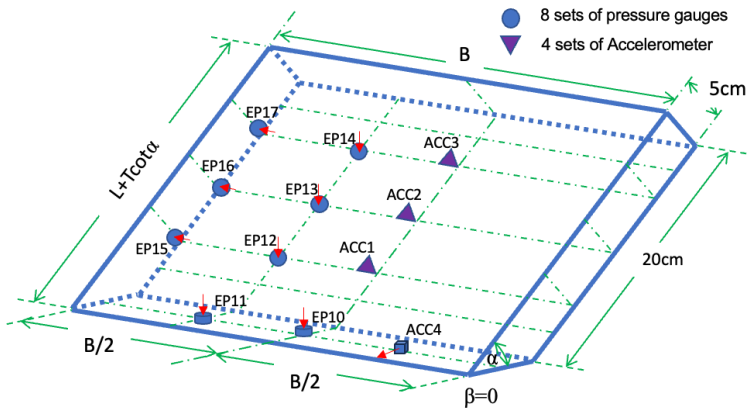


Figure 8-4 Slope model with instrumentations.



Figure 8-5 Soil chamber and model (model scale)

Table 8-1 Material properties of Edosaki sand and Physical properties
(Aroonwattanaskul, 2020)

Water content (w)	8%
Maximum dry density ($\rho_d \text{ max}$)	1700 kg/m ³
Unconfined compressive strength (σ_c)	5.75 kN/m ²
Degree of compaction (D_c)	90%
Internal friction angle (ϕ)	44.2°
Cohesion (c)	1.21 kPa
Interface friction angle of humid sand on Aluminum(ϕ_i)	25.4°
Interface adhesion of humid sand on Aluminum (c_i)	0.233 kPa
Interface friction angle of humid sand on Teflon sheet(ϕ_t)	19.01°
Interface adhesion of humid sand on Teflon sheet (c_t)	0.08 kPa

8.3 Experimental procedures

Once all the gauges had stabilized under 20G, the model was subjected to a sequence of four seismic excitations. The amplitudes of these waves were incrementally increased, following a reference ramped sine wave pattern (as illustrated in Figure 8-6): The sine wave had an amplitude of 0.05 m and a frequency of 1 Hz, mimicking a duration of 30 seconds in the prototype scale.

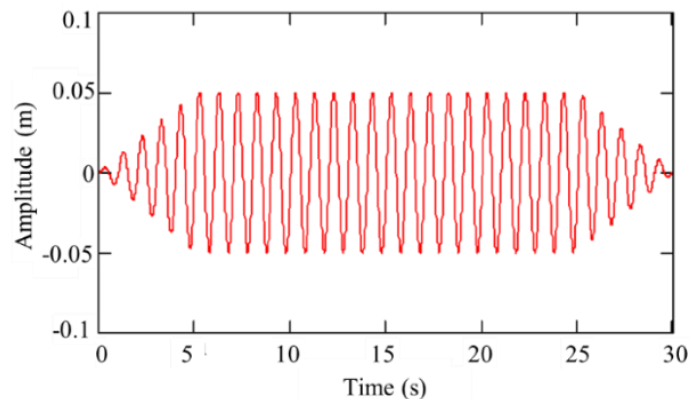


Figure 8-6 Datum input wave (prototype scale)

The amplified values are shown in Table 8-2. Throughout the course of the experiment, the acceleration was captured by accelerometers embedded into the model. The highest recorded value from ACC4, positioned at the base of the model, was adopted to signify the ground motion. This value was subsequently converted into a horizontal seismic coefficient for each shaking event.

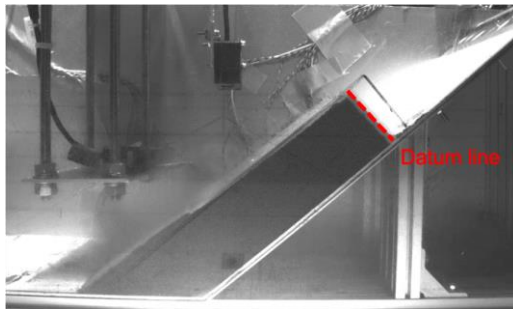
Table 8-2 Fourth instances of seismic waves (prototype scale)

Number	1	2	3	4
Magnified number	0.2	0.4	0.6	0.8
k_h	0.096	0.168	0.242	0.329

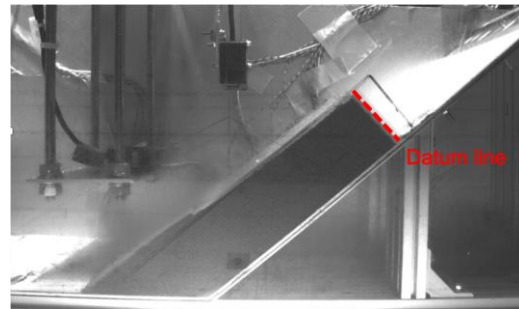
8.4 Experimental results

8.4.1 Failure observations

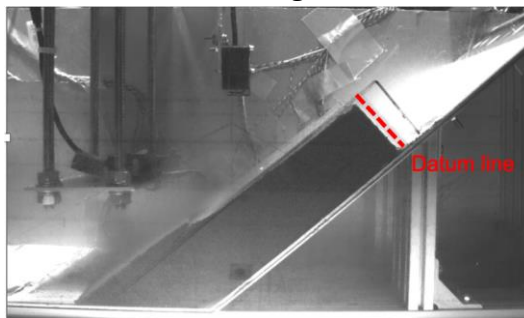
The visual documentation of the sliding process was captured employing a high-speed camera. The arrangement of the slope model subsequent to shaking is portrayed in Figure 8-7. Following the attainment of a centrifugal acceleration of 20G, the slope model was set as the initial condition, with the datum line marked in Figure 8-7(a). Upon the application of the first shaking, the slope model retained its stability (see Figure 8-7(b)). Subsequent to the second shaking, a minor downward displacement along the bedding plane was observed (see Figure 8-7(c)). The third shaking led to an upheaval at the slope's toe and deformation at its crest (see Figure 8-7(d)). Upon the implementation of the fourth shaking, a more pronounced toe sliding, and a larger overall slope movement were observed compared to earlier stages (see Figure 8-7(e)).



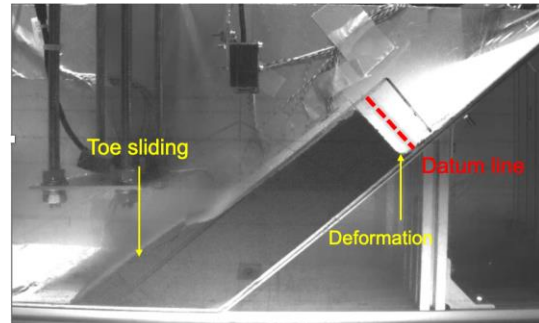
(a) Slope model after loaded acceleration to 20g.



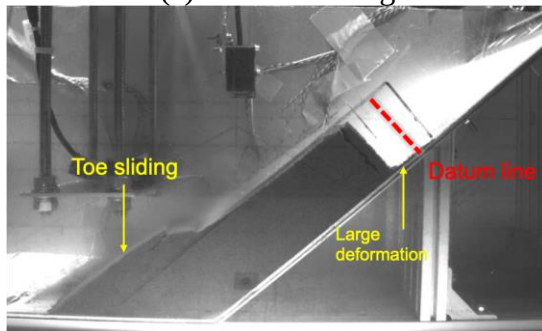
(b) the 1st shaking



(c) the 2nd shaking



(d) the 3rd shaking



(e) the 4th shaking

Figure 8-7 Captured image from video high-speed camera

8.4.2 Displacement

The displacement of the slope was tracked using laser sensors capable of detecting movement along both vertical and horizontal directions. The outcomes derived from these laser sensors are depicted in Figure 8-8. During the first shaking, the slope model exhibited a minor displacement in both vertical and horizontal directions. Subsequent to the second shaking, vertical movement displayed no noticeable change, while horizontal movement became more pronounced. During the third shaking, a similar pattern emerged to that of the second shaking, albeit with slightly larger horizontal movement. With the fourth shaking, the movements displayed a significant change in both vertical and horizontal directions, showcasing an obvious shift in behavior.

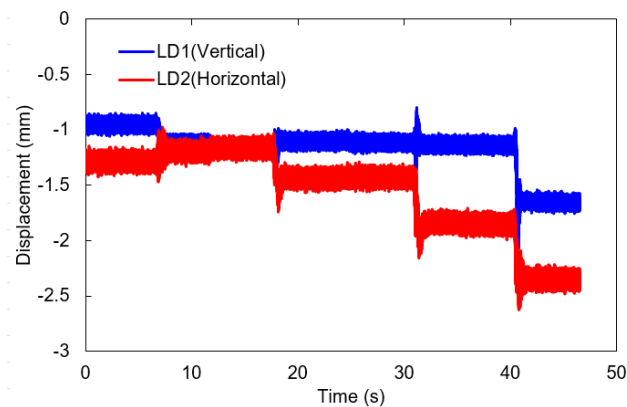


Figure 8-8 Slope displacement versus time (prototype scale).

8.4.3 Earth pressure

As specified, a total of 5 earth pressure gauges (EP10, EP11, EP12, EP13, EP14) were affixed to the soil chamber to monitor horizontal earth pressures. It's noteworthy that EP10 did not respond upon reaching the centrifugal acceleration of 20G. The results of earth pressure are illustrated in Figure 8-9. Among the gauges, EP11, EP12, and EP13 exhibited slight changes during the application of the first shaking, while EP14 exhibited an immediate drop in readings. The second shaking yielded similar earth pressure patterns as the first shaking across all gauges. Changes in earth pressure became evident during the third shaking, with EP11 displaying a dramatic increase. When the fourth shaking was applied, EP11, EP12, and EP13 showed a similar increase as in the third shaking, while EP14 exhibited a more moderate increase. The data obtained from the earth pressure gauges affirm the significant changes in the slope model when applied third shaking. This is consistent with the observable results captured through imagery, suggesting a significant alteration likely due to slope sliding.

In accordance with the reduction factor (r_d) (Pipatpongsa, Fang, Leelasukseree, and Chaiwan, 2022) can be determined through Eq. (8-23). This analysis incorporates both the effective horizontal stress (σ'_h) and effective vertical stress (σ'_v) at the same level as essential components. For the evaluation of the reduction factor, three pairs of earth pressure readings are employed: EP12 and EP15 (r_{d1} , near the toe support), EP13 and EP16 (r_{d2} , middle of the slope), and EP14 and EP17 (r_{d3} , crest of the slope). The evolution of r_d during each shaking is depicted in Figure 8-10. Notably, r_{d1} demonstrates an immediate increase during each shaking due to slope failure occurring in a passive state ($r_d > 0$). Similarly, r_{d2} experiences an immediate rise upon the application of shaking. In contrast, r_{d3} commences at around -1 and gradually approaches zero during shaking, indicative of the limited influence of soil weight on the crest of the slope.

$$r_d = \frac{\sigma'_h - \sigma'_v}{2\sqrt{(\sigma'_v \tan \phi + c)(\sigma'_h \tan \phi + c)}} \quad (8-23)$$

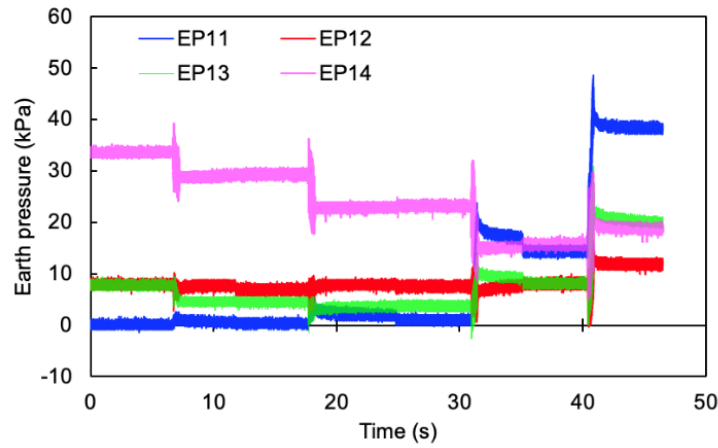


Figure 8-9 Recorded earth pressure during the application of shaking (prototype scale).

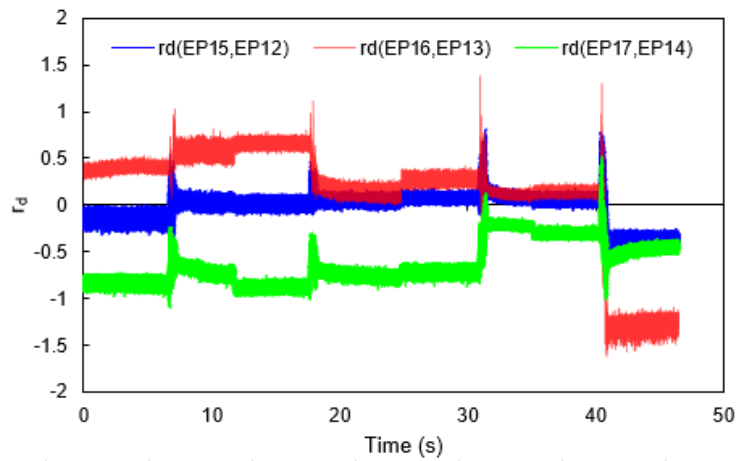


Figure 8-10 Reduction factor during fourth shaking

Table 8-3 Reduction ratio during shaking (prototype scale).

Wave		Maximum value during shaking		
No.	k_h	r_{d1} (EP15 ,EP12)	r_{d2} (EP16, EP13)	r_{d3} (EP17, EP14)
1st	0.096	0.54	1.03	-0.24
2nd	0.168	0.46	1.11	-0.29
3rd	0.242	0.81	1.37	0.13
4th	0.329	0.77	1.30	0.52

8.5 Discussions

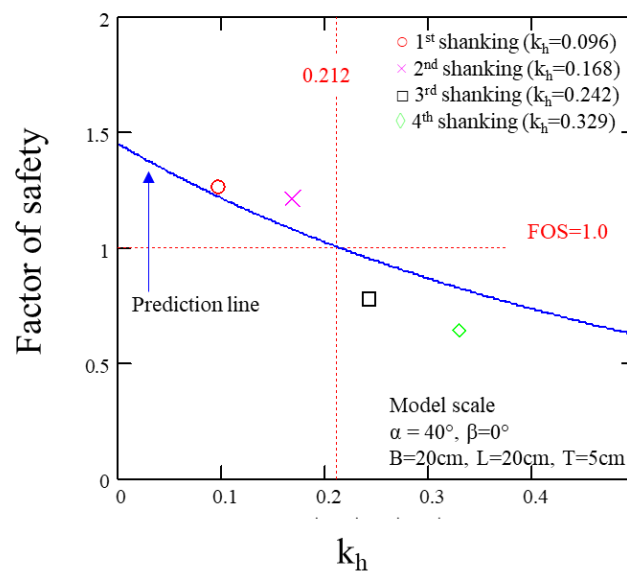


Figure 8-11 Comparison between theoretical prediction and experimental results

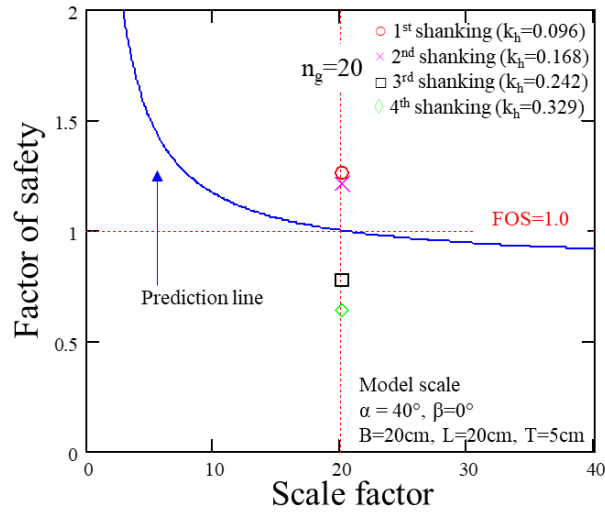


Figure 8-12 Comparison between theoretical prediction and experimental results

Based on the captured images that feature the initial datum line alongside the earth pressure measurements, it revealed that the slope exhibited potential for sliding during the application of the third shaking ($k_h=0.242$). Subsequently, it was feasible to perform back-calculations of the factor of safety (FS) by utilizing the values of k_h and the shear reduction ratio (r_d) corresponding to each shaking event. This was then juxtaposed against the theoretical predictions for comparison. Table 8-3 displays the maximum reduction ratios observed during shaking. It is noteworthy that for theoretical calculations, slope failure corresponds to $r_d=1.0$. Thus, r_d values have been selected from critical states, approximating 1.0. Specifically, the reduction factors for the respective shakings are as follows: first shaking $r_d=1.03$, second shaking $r_d=1.11$, third shaking $r_d=0.81$, and fourth shaking $r_d=0.77$.

The comparison between the experimental findings and the theoretical prediction is presented in Figure 8-11. Specifically, the theoretical estimation of k_h that aligns with the point of slope failure is marked at 0.212 (resulting in $FS=1.0$). In terms of factor of safety, the calculated FS during the first shaking ($k_h=0.096$) amounted to 1.259, during the second shaking ($k_h=0.168$) it was 0.774, during the third shaking ($k_h=0.242$) it stood at 0.994, and during the fourth shaking ($k_h=0.329$) it decreased to 0.642. Figure 8-12 illustrates the comparison under 20G shaking conditions, revealing that lower values of k_h exhibit greater stability in comparison to higher k_h values.

This comparative analysis underscores a remarkable congruence between the theoretical predictions and the experimental observations. The theoretical predictions of dip slope stability closely mirror the experimental results, attesting to the efficacy and reliability of the applied methodologies.

8.6 Summary

In summary, this investigation underscores the efficacy of combining dynamic centrifugal modeling with pseudo-static stability analysis for evaluating the seismic resilience of laterally confined dip slopes. Through rigorous controlled experiments and theoretical analyses, we established a remarkable congruence between predicted and observed outcomes:

(1) Evident slope movement after the application of the third shaking was deduced from captured images featuring the initial datum line.

(2) Empirical earth pressure findings validated slope failure during the third shaking.

(3) While horizontal laser sensors effectively detected slope movement during the third shaking, the lack of vertical displacement, potentially attributed to their middle slope positioning, is noteworthy.

(4) Theoretical predictions closely align with the experimental findings, as evident from the estimation of $FS=1.0$ when applying a k_h value of 0.212, which closely corresponds to the third shaking event where $k_h=0.242$.

Chapter 9 Conclusions and Future work

The current research introduces novel approaches to investigate the failure mechanisms of dip slopes. Physical modeling techniques were employed to simulate dip slope behavior triggered by both moving boundaries and groundwater infiltration. Furthermore, dynamic centrifuge physical models were utilized to analyze the impact of seismic loading on dip slopes. To classify the failure modes of dip slopes accurately, a PIV analysis with a statistical approach was developed. Additionally, theoretical calculations of slope stability were enhanced by incorporating concepts of lateral confinement and groundwater seepage. These combined methodologies offer a comprehensive understanding of dip slope behavior and contribute to the advancement of slope stability analysis.

In **Chapter 2**, The chapter reviews prior research on dip slopes and their triggers, aiming to integrate insights into this study's framework. It adapts and extends established methodologies to enhance understanding and develops innovative solutions for addressing research objectives:

- (1) Study the reverse toe effect on failure mechanisms of dip slope by physical modelling
- (2) Development of PIV analysis with statistical approach to evaluate dip slope failure mechanisms
- (3) Investigate the effect of groundwater infiltration on dip slopes by centrifugal modelling
- (4) Explore counterweight mitigation for dip slope stabilization by centrifugal modelling
- (5) Study effect of dynamic loading on dip slope by centrifugal modelling
- (6) Development and validation of slope stability analysis formula

In **Chapter 4**, the reverse toe effect on the failure mechanism of dip slopes was studied using 1g physical modeling. Physical models were constructed with various reverse toe angles and were triggered by a surcharge load cart, serving as a large and moving boundary. Particle Image Velocimetry (PIV) was applied to analyze movements during slope failure. Additionally, the minimum coefficient of variation (min CV) was used to detect slip line formation by analyzing the velocity vector field from PIV results.

The findings from the PIV analysis, incorporating min CV, reveal that the reverse toe angle significantly affects the failure mode. Two distinct failure modes were identified: toe sliding, where the entire slope moves during collapse, and thrust failure, where only part of the slope moves, and upheaval occurs. Moreover, PIV analysis with minimum coefficient of variation (CV) can be used to calculate the failure time by comparing it with the inverse velocity method. The results reveal that the time of minimum CV closely matches the predicted time of failure from the inverse velocity method.

In **Chapter 5**, the effect of groundwater infiltration on dip slopes was investigated using centrifugal modeling. An impermeable bedding plane was chosen to simulate parallel groundwater seepage. Based on the failure mode findings in Chapter 4, two series of slope geometries were designed. Two triggering methods were used: a surcharge load and groundwater infiltration. The experimental results revealed that slopes triggered by both surcharge load and groundwater experienced toe sliding. Conversely, slopes triggered by surcharge load alone exhibited both toe sliding and thrust failure. Additionally, the applied force measurements showed that slopes without groundwater required more force to trigger failure compared to slopes with groundwater. Thus, groundwater infiltration significantly decreases slope stability.

Moreover, this chapter provided theoretical calculations of slope stability based on the limit equilibrium method, incorporating lateral confinement and groundwater infiltration. The Factor of Safety (FOS) was used to indicate slope stability during experimental procedures. Experimental results were utilized to validate the theoretical calculations. The reduction ratio (r_d) played an important role in determining lateral resistance, and for predicting slope failure, r_d was assumed. The comparison between experimental results and predicted FOS revealed that the predicted FOS decreased during slope failure, aligning with the experimental observations.

In **Chapter 6**, centrifugal modeling was designed based on the findings from Chapter 5, where the failure mode was primarily toe sliding. To address this issue, counterweight fill was used to mitigate slope instability. The experimental results revealed that sufficient counterweight fill is shift failure mode from toe sliding to thrust failure.

Moreover, this chapter develops the theoretical calculations from Chapter 5 by incorporating counterweight fill. This addition increases toe resistance (P_S). When P_S increases to equal the passive resistance limit (P_T), the failure mode changes from toe sliding to thrust failure. Thus, the failure mode depends on the minimum value between P_S and P_T . Moreover, the optimum counterweight (Q_P) can be calculated based on P_S equaling P_T . The validity of the optimum counterweight was confirmed through experimental results, which demonstrated that toe sliding occurred when the test

counterweight was lower than the optimum counterweight, while thrust failure occurred when the test counterweight was higher than the optimum counterweight.

In **Chapter 7**, a laterally confined dip slope situated on a permeable bedding plane underwent study via centrifugal modeling, with theoretical calculations adapted from Chapters 5 and Chapter 6. However, the preceding theoretical calculations were based on the concept of parallel groundwater seepage, necessitating adjustments to the groundwater seepage model. Two experiment series were conducted: slope without counterweight and slope with counterweight. Experimental results revealed that the slope without counterweight fill experienced toe sliding. Moreover, slopes with sufficient counterweight fill could mitigate slope failure. These experimental results align with findings from Chapter 5 and Chapter 6. However, the predicted Factor of Safety (FOS) by assuming a reduction ratio (r_d) indicated slope stability, contrasting with the experimental FOS. The assumption of r_d used in previous chapters cannot be applied here due to changes in groundwater seepage. Further investigation into this issue is warranted.

Chapter 8 investigates the seismic resilience of laterally confined dip slopes through a combination of dynamic centrifugal modeling and pseudo-static stability analysis. The findings suggest that seismic activity substantially reduces slope stability, resulting in slope failure. Theoretical predictions closely correspond with experimental observations, affirming the efficacy of the proposed approach. However, further experimental tests are required to validate the pseudo-static stability analysis comprehensively.

Limitations of the Research: The experiments conducted on both 1g and centrifugal physical models utilized a surcharge cart as a large boundary. The soil model, confined by lateral supports, moved symmetrically along the bedding plane when the surcharge was applied, aligning with theoretical calculations that assume sliding along the bedding plane. Unlike circular slip failures in homogeneous soils, the failures examined in this study involve sliding along a pre-existing weak plane, such as a bedding plane, which is typical in stratified geological formations. The failure in these scenarios is planar, governed by the bedding plane and lateral confinements, rather than circular slip failure.

For engineering design, 2-dimensional analysis is commonly used in slope stability assessments when the slope's breadth is unknown. However, incorporating lateral confinements into the analysis allows for a more comprehensive 3-dimensional approach. This research utilized 3D analysis to examine experimental results, validate theoretical calculations, and define dip slope failure mechanisms. While 3D analysis is

particularly effective under controlled experimental conditions, real-world applications, boundary conditions often exhibit variability, inhomogeneity, and especially unknown failure breadths, which can limit the generalizability of findings.

Despite these challenges, the research findings remain applicable to specific sites with conditions similar to those studied. For instance, the Mae Moh open-pit mine was selected for analysis due to its resemblance to the research conditions. Although the results are especially relevant to the Mae Moh mine, they can also be applied to other scenarios with comparable criteria. Therefore, this research offers valuable insights for engineering practice and can be utilized for future projects with similar conditions.

Finally, the recommendations for future studies are summarized as follows:

- (1) The PIV analysis with the minimum coefficient of variation aims to investigate slip lines and calculate failure time from a side view. However, in engineering practice, instruments can only monitor surface displacement and velocity. Therefore, a physical model is required to investigate surface movement in relation to site monitoring.
- (2) Due to the influence of groundwater infiltration on slope stability analysis, slopes situated on impermeable bedding planes can be assessed using the concept of slope stability with parallel seepage. However, for slopes on permeable bedding planes, groundwater seepage accumulates at the toe, and predicted factors of safety using adjusted methods fail to accurately predict slope failure. Therefore, there is a critical need to enhance theoretical calculations in order to improve accuracy in such scenarios.
- (3) Pseudo-static stability analysis was developed for dip slopes in Chapter 8. The experimental results aligned with theoretical calculations. However, the experiment was conducted for only one case. Therefore, to validate these calculations, the experiment needs to be repeated.

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