

Physical Modelling of Dam Overtopping from Landslide Generated Waves

By: Netsanet Nigatu Tessema

Addis Ababa University

December, 2020



Addis Ababa University
Addis Ababa Institute of Technology
School of Civil and Environmental Engineering

Physical Modelling of Dam Overtopping from Landslide
Generated Waves

A thesis submitted to the school of Graduate studies of Addis Ababa University in Partial fulfillment of the degree of Master of Science in Civil Engineering (Hydraulic Engineering Stream).

By:

Netsanet Nigatu Tessema

Advisor:

Asie Kemal Jabir (Dr. Ing.)

December, 2020

Addis Ababa University
Addis Ababa Institute of Technology
School of Graduate studies
School of Civil and Environmental Engineering

Physical Modelling of Dam Overtopping from Landslide Generated Waves

This is to certify that the thesis prepared by Netsanet Nigatu Tessema entitled ‘**Physical modelling of dam overtopping from landslide generated waves**’ and submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering under Hydraulic Engineering compiles with the regulations of the university and meets the accepted standards with respect to originality and quality.

By:

Netsanet Nigatu Tessema

Approval by Board of Examiners

Dr. Mebruk Mohammed

Chairman (Department of Graduate Committee)

Signature

Dr. Ing. Asie Kemal

Advisor

Signature

Dr. Daniel F/Selassie

Internal Examiner

Signature

Dr. Yilma Sileshi

External Examiner

Signature

Certification

The undersigned certify that he has read the thesis entitled: **Physical modelling of dam overtopping from landslide generated waves** and hereby recommend for acceptance by the Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science.

January 1, 2021

Dr. Ing. Asie Kemal (Advisor)

Date

This thesis is dedicated to my mom 'Tsehay T/Mariam'.

Acknowledgment

First, I like to express my deepest gratitude to my supervisor Dr. Ing Asie Kemal Jabir, for believing in me and being optimist all the time. I am sincerely grateful to Professor Leif Lia and Asst. Professor Fjola Siggtrydoitr for inviting me to NTNU and allow me to conduct the experiments on the physical model in Hydraulic laboratory of NTNU. Furthermore, many thanks go to the Dean of school of Civil and Environmental Engineering, Dr. Mebruk Mohammed for the cooperation and guidance you provided me with. Finally, I would like to express my profound gratitude to my families for providing me with endless support and encouragement all the way.

Abstract

Landslide generated waves may have disastrous impact on the surrounding environment. The catastrophic event such as Vajont disaster occurred in Italy, 1963 and caused the loss of more than 2000 human lives, is among the reasons that landslide generated wave and consequences is of great deal. This study is based on a physical model test in laboratory of Hydraulic Engineering at NTNU which simulates a landslide released, fall into reservoir, propagates and overtops a dam. This thesis is a parametrical study on landslide generated impulse waves based on experiments conducted in a laboratory on a 1: 190 scale model. The model has three major parts: the slide, the reservoir and the dam. The slides are simulated with rigid blocks sliding over the slide ramp, falling into reservoir, generating wave, runs up and overtops a dam. All the parameters are measured with different sensors installed.

To investigate the parameters which define the wave generation and overtopping of a dam due to slide generated waves, a systematic variation of parameters like slide volume, slide release height, still water depth and upstream dam slope, were applied. Based on the results from this study, the slide volume is found to be the most dominant parameter that defines the overtopping volume due to impulse waves. In addition, freeboard is found to be another parameter that has significant impact on overtopping volume. For a fixed setup, an increase in freeboard decreases the overtopping volume. The other parameter investigated is the slide impact velocity, where results show that the slide release height is the most dominant factor to define it. The most important part of the study is that the overtopping volume and the overtopping depth is not uniform along the five dam crest sections, where the maximum is recorded with the ultrasonic sensors installed at the edges namely, CH11 and CH16. Finally, it is observed that it is difficult to apply the results from the literature to this study due to the different model setup used in this study.

Keywords

Impulse waves, Landslide, wave generation, wave propagation, overtopping, Impulse product parameter.

Table of Contents

Acknowledgment	I
Abstract	II
Keywords	III
Table of Contents	IV
List of Figures	VI
List of Tables	IX
List of Abbreviations	X
1. Introduction	1
1.1. General	1
1.2. Statement of the problem	2
1.3. Scope of the study	5
1.4. Objectives	6
1.5. Research questions.....	6
1.6. Significance of the study.....	7
1.7. Outline of the thesis	7
2. Theoretical Background	8
2.1. Overview	8
2.2. Wave generation and propagation.....	12
2.3. Slide movement type.....	16
2.4. Wave runup and overtopping	17
2.5. Historical impulse waves in reservoirs	20
3. Physical Model	23
3.1. Experimental setup.....	23
3.1.1. The slide.....	24
3.1.2. The reservoir	26
3.1.3. The dam	27
3.2. Measuring Instruments.....	29
3.3. Instrument calibration	32
3.4. Froude's Model law	34
4. Test Procedure and Data Reliability	37
4.1. Experimental procedure	37
4.2. Data reliability	39
4.2.1. Normal distribution by Graph	40

4.2.2.	Kolmogorov-Smirnov test for Normality.....	41
4.2.3.	Q-Q Plots (quantile-quantile plot).....	43
4.2.4.	Box plots	45
5.	Result and Discussion	49
5.1.	Wave propagation and overtopping pattern	49
5.2.	Slide impact velocity.....	53
5.3.	Overtopping volume	55
5.4.	Comparison to literature.....	58
6.	Conclusion and Recommendation	63
6.1.	Conclusion	63
6.2.	Recommendation	64
References		66

List of Figures

Figure 1: Sketch map of Ethiopia showing the localities where landslide have been reported (Abebe et al., 2010).	3
Figure 2: Gilgel Gibe catchment with landslide scar (Broothaerts et al., 2012).	3
Figure 3: View of Tekeze dam site looking downstream; area of potential instability indicated in shape to the left and toppling condition, upper left abutment above dam to the right.	4
Figure 4: Wave generation mechanisms with subaerial, partially submerged, and underwater slide (Heller, 2007).	8
Figure 5: The three impulse wave phases (Heller, 2007).	9
Figure 6: Characteristics of Stokes wave (Heller et al., 2009).	10
Figure 7: Characteristics of Cnoidal wave (Heller et al., 2009).	10
Figure 8: Characteristics of Solitary wave (Heller et al., 2009).	11
Figure 9: Characteristics Bore waves (Heller et al., 2009).	11
Figure 10: Sketch defining the governing parameters on impulse wave generation and the most important wave parameters (Heller et al., 2009).	14
Figure 11: Mass movement types: (a) sliding, (b) flowing, (c) falling, (d) toppling, and (e) spreading (Cruden and Varnes, 1996).	16
Figure 12: Sketch defining the governing parameters for the third phase, wave run up and dam overtopping (Heller et al., 2009).	18
Figure 13: (a) Pre landslide view looking east over the dam in foreground, (b) Post slide view looking east, and (c) North looking view of the Piave valley after the flood (Steven and Simon, 2011).	21
Figure 14: Picture of the physical model used for this study.	23
Figure 15: The 3D view of the model setup used for this study.	24
Figure 16: The rigid blocks used in this study parallelopiped block at the left and rectangular block at the right.	25
Figure 17: Picture showing different block configurations used in this study.	26
Figure 18: Picture of the reservoir in the laboratory.	27
Figure 19: Section view of the dam.	28
Figure 20: Picture of the dam crest divided into five sections.	28
Figure 21: Picture of the five buckets collecting the overtopping water over the five dam crest sections.	28
Figure 22: Placement of all the sensors used in this study.	29
Figure 23: The picture of five ultrasonic sensors for measuring the overtopping depth along the five dam crest sections.	30

Figure 24: Ultrasonic sensor used for measuring the overtopping volume over the dam crest in each bucket.	31
Figure 25: Picture of rotational sensor for measuring the slide speed.	31
Figure 26: A computer with Agilent software.	33
Figure 27: Picture of a Piezometer used for measuring the reservoir water level.	37
Figure 28: Plot between a histogram and normal distribution curve for CH5, CH12 and the total overtopping volume in the buckets.	41
Figure 29: QQ plot between the theoretical and actual sample data for CH5, CH12 and crest 3.	44
Figure 30: Box plot for the maximum wave height recorded with CH1 to CH9.	46
Figure 31: Box plot showing the outliers for maximum overtopping depth recorded with CH11 to CH16.	46
Figure 32: Box plot showing the outliers of the total overtopping volume measured in the five buckets.	47
Figure 33: The plot between theoretical and actual sample data before and after removal of outliers for CH1, CH2 and CH16.	47
Figure 34: Normal distribution plot for CH4.	48
Figure 35: Plot for the wave height vs time for CH4.	48
Figure 36: Plot for wave height recorded at the 1 st row (CH1, CH4 and CH7) versus time (Test no. 288_2.25s_4.5_2H_150).	50
Figure 37: Plot for wave height recorded at the 2 nd row (CH2, CH5 and CH8) versus time (Test no. 288_2.25s_4.5_2H_150).	50
Figure 38: Plot for wave height recorded at the 3 rd row (CH3, CH6 and CH9) versus time (Test no. 288_2.25s_4.5_2H_150).	51
Figure 39: The overtopping pattern over the five dam crest sections (Test no. 188_2.25s_4.5m_2H_150).	52
Figure 40: Histogram for overtopping depth distribution over the dam crest (Test no. 188_2.25s_4.5m_2H_150).	52
Figure 41: Average versus corrected speed for 203_2.25s_4.5_4_150.	54
Figure 42: Slide velocity comparison considering similar setup for different release height. The test numbers considered are: 185_2.25s_4.5_2H_200, 188_2.25s_4.5_2H_150 and 191_2.25s_4.5_2H_100.	54
Figure 43: Overtopping volume distribution over the dam crest (Test no. 188_2.25s_4.5m_2H_150).	55
Figure 44: Overtopping volume for different slide arrangements ($\beta = 1: 1.5$ and $h_o = 0.5$ m).	56
Figure 45: The figure shows the impact of slide volume and freeboard on overtopping volume.	57

Figure 46: The relationship between slide volume and slide release height for 2V slide block arrangement.....	58
Figure 47: The comparison between the measured and predicted (with Eq. (2)) maximum amplitude.	59
Figure 48: The figure shows the comparison of measured relative overtopping volume versus P.....	60
Figure 49: The comparison of measured and predicted relative overtopping volume.	61
Figure 50: The comparison of measured and predicted (with Kobel's et al. (2017) Eq. (12)) relative overtopping depth.	61

List of Tables

Table 1: Limitations for the calculation of the impulse wave generation (Heller et al., 2009).....	15
Table 2: Some historical subaerial and partially submerged landslide generated impulse waves (Heller, 2007).....	22
Table 3: Slide block characteristics.	25
Table 4: Types of sensors used for the study	29
Table 5: Similarity types between a model and a prototype. The subscripts, m and p, stand for model and prototype.	34
Table 6: list of forces for landslide generated impulse waves.	35
Table 7: The most common scale ratios for scaled model with Froude's model law.....	35
Table 8: Summary of performed experiments in this study.....	38
Table 9: Parameters for normal distribution with KS test considering wave heights.	42
Table 10: Parameters for normal distribution with KS test considering the maximum overtopping depth.....	43
Table 11: Parameters for normal distribution with KS test considering the total overtopping volume.	43
Table 12: Parameters for box plot in investigating the outliers considering the maximum wave height.	45

List of Abbreviations

a (m) = wave amplitude;

a_M (m) = maximum wave amplitude;

b (m) = slide width;

b_b (m) = reservoir bottom width;

B_c (m) = crest width;

C_d (-) = coefficient of discharge;

d_o (m) = maximum overtopping depth;

d_{max} (m) = maximum overtopping depth;

E (-) = overtopping volume parameter;

f (m) = freeboard;

F (m) = Froude number;

g (m/s^2) = gravitational acceleration;

h (m) = still water depth;

h_o (m) = landslide release height;

H (m) = dam height;

H_M (m) = maximum wave height;

L (m) = wave length;

l_b (m) = reservoir length;

l_c (m) = crest length;

l_s (m) = slide length;

M (-) = relative slide mass;

m_s (Kg) = slide mass;

s (m) = slide thickness;

S (-) = relative slide thickness;

t (s) = time;
 T (s) = wave period;
 T (-) = wave type product;
 P (-) = impulse product parameter;
 q (m³/s/m) = the mean overtopping discharge per meter;
 Q_{max} (m³/s) = maximum discharge;
 R (m) = run up height;
 R^2 = Coefficient of determination;
 t (s) = overtopping duration;
 v_s (m/s) = slide impact velocity;
 W (-) = runup height parameter;
 W_s (m³) = slide volume;
 W_w (m³) = overtopping volume;
 α (°) = slide ramp inclination;
 Φ (°) = slide front angle;
 Φ_s (°) = internal friction angle;
 θ (°) = slope of side banks;
 β (°) = upstream dam slope angle;
 ε (-) = relative wave amplitude;
 ρ_s (kg/m³) = bulk slide density;
 ρ_w (kg/m³) = density of water.

1. Introduction

1.1. General

Impulse waves can occur in open oceans, bays, lakes and reservoirs as the result of landslides, rockfalls, shore instabilities, avalanches or glacier calving. They are classed as gravity waves and can, in extreme cases, result in the overtopping of dams, with catastrophic consequences. The potential damage as a result of overtopping water is more serious for embankment dams. Alpine regions face a high risk of such events in view of their steep valley flanks, their potentially large slide volumes, with high impact velocities, and their large number of reservoirs (Heller, 2007). Overall 20,000 casualties are attributed to the destructive power of mass flow generated impulse waves (Slingerland and Voight, 1979). Among these catastrophes one of them occurred in 1958, in the Lituya Bay, Alaska, where a landslide generated wave reached a maximum run up height of 524 m (Miller, 1960). Another example occurred in Vaiont, North Italy in 1963. It was the most destructive impulse wave in history where a slide generated wave overtopped the concrete arch dam by more than 70 m destroying the village Longarone (about 2,000 people were perished). The arch dam remained practically undamaged (Genevois and Ghirotti, 2005). These events show that this threat is real and understanding of the phenomenon is crucial.

Passive methods are mainly applied to prevent damages due to impulse waves such as evacuation, water level draw down in artificial reservoirs, freeboard control, or blasting of possible slides. These methods require a detailed knowledge of the wave features such as the wave height or the wave decay (Heller and Hager, 2010). Since these parameters are currently not satisfactorily predictable, the water level of many artificial reservoirs is kept well below the maximum possible level (Panizzo et al., 2005). Consequently, in landslide prone areas, the

dam and reservoir potentials are underutilized. These drawbacks led to the investigation of the topic landslide generated waves overtopping a dam.

In 2008, the laboratory of the department of Hydraulic Engineering at NTNU, Trondheim built a physical model scale model of Storfjord to predict the wave height due to the Åkerneset landslide. Once the tests were finished, the model was transformed into a generic model of a reservoir with a lateral landslide. The physical model simulates landslide falling perpendicular to reservoir's longitudinal axis, generating wave which propagates along the basin and overtops the dam. Many researches have been conducted on the model since then. This study is a continuation of these studies.

1.2. Statement of the problem

Landslide hazard is one of the crucial environmental constraints for the development of Ethiopia, representing a limiting factor for urbanization and infrastructural projects and generally, for all the activities performed on and at the foot of slopes (Abebe et al., 2010). The damage produced by landslides in Ethiopia is relevant from 1993 to 1998 alone, more than 200 houses were destroyed, more than 500 km of roads were interrupted and about 300 people were killed (Ayalew, 1999). The hilly and mountainous terrains of Ethiopia are characterized by variable topographical, geological, hydrological and lands use conditions. The northern Omo river basin, the lower Wabe shebelle river valley, the Wendo Genet slope, the Blue Nile gorge, the town of Dessie, the Wudmen area in Weldiya, the Gilgel Gibe river, the Uba Dema village in Sawla, and parts of Tigray are some of the areas where imposing landslide events have been seen (Abebe et al., 2010) (see Figure 1).

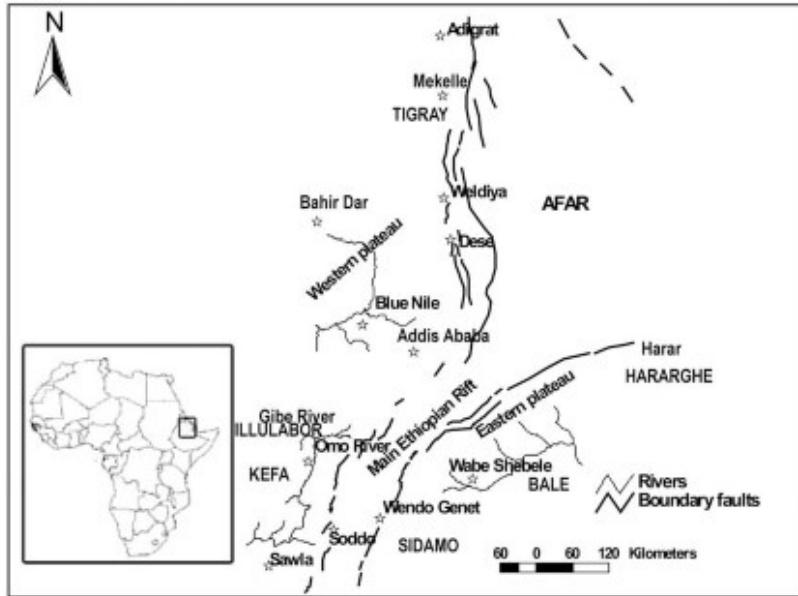


Figure 1: Sketch map of Ethiopia showing the localities where landslide have been reported (Abebe et al., 2010).

Although most of the dams in Ethiopia are constructed on reservoirs with steep slopes, slides impacting a reservoir and dam are not extensively studied. However, there are examples of slides falling into reservoir in Ethiopia.

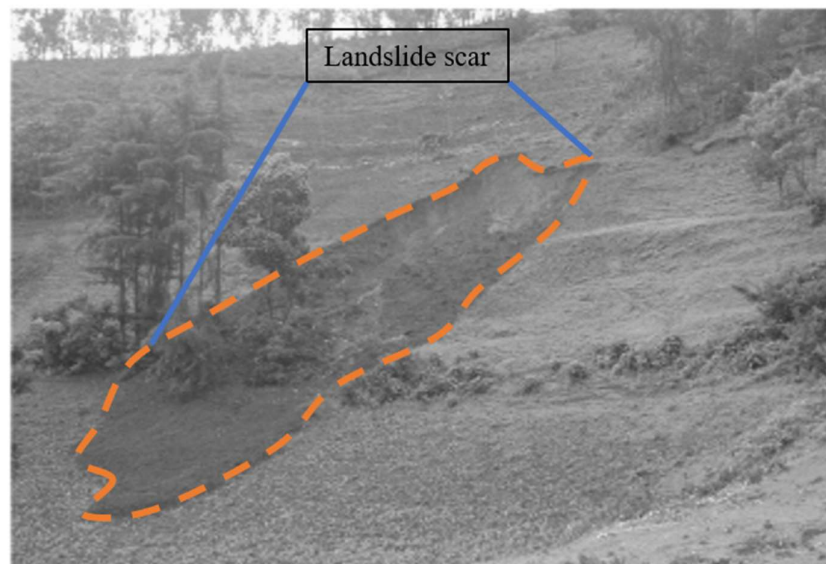


Figure 2: Gilgel Gibe catchment with landslide scar (Broothaerts et al., 2012).

One example is the Gilgel Gibe catchment where landslides falling into reservoir are recorded as a source of sediment in Gilgel Gibe I hydroelectric power plant (Broothaerts et al., 2012). Figure 2 shows a landslide scar at Gilgel Gibe catchment having an area of 700 m² and landslide volume of 1400 m³.

Another example is the Tekeze arch dam, which is found to be prone to landslide triggered impulse waves because of steep reservoir boundary, narrow reservoir geometry, possible large slide masses, and high impact velocities (Aman et al., 2019). Accordingly, on April 2008 massive slide occurred near Tekeze dam site. There were creeping slides along the reservoir gorges, hence a retaining wall for protecting the slide from falling into reservoir has been built. These has led to an additional cost of 42 million dollars (Hailemariam Gugsu and Schneider, 2010). There were also a major joint subparallel to the slope that, combined with the effects of weathering, contributing to a serious and unstable toppling conditions (Dickson and Loar, T.N., 2011) (Figure 3). Hence, these landslides and landslide generated slope failures into reservoir are inevitable, due attention has to be given in order to reduce losses from such hazards.



Figure 3: View of Tekeze dam site looking downstream; area of potential instability indicated in shape to the left and toppling condition, upper left abutment above dam to the right.

The impact of landslide generated waves on the design of dams is not explicitly studied in Ethiopia. The national guideline of dam prepared by ENTRO in the year 2013, considers wind

setup and wave runup as a principal design criterion for freeboard and discusses in brief the calculation procedure of these factors. However, landslide generated waves should be considered in the design of new dams and analysis for the impact of impulse wave has to be made for existing dams. There is a recent study by Aman et al. (2019), which investigates the impact of landslide generated wave triggered by earthquake on Tekeze dam based on the simplified equations given by (Heller, 2007) and (Müller, 1995).

The equations describing the impulse wave generated waves are mostly derived based on physical model test on rectangular flume where the slide impacts directly to the reservoir and then the dam. However, this study is based on the experimental results from a physical model test, where the slide impinges into reservoir's longitudinal axis. Hence, this study gives a basic insight to topic of the landslide generated waves which can be further studied presenting the dominant parameters that describes the phenomenon. The outcomes from this study can be used as an input in developing a general predictive equation which can be used in dam design against these landslides generated waves. Knowing these further helps in taking preventive measures and also emergency preparedness once these threats are found in the area.

1.3. Scope of the study

The impact of landslide generated waves on dams is investigated with a physical model. This is a parametric study which investigate the parameters on the slide impact and overtopping zone, which can further be used as an input in further detailed analysis. The model setup describes the landslide impacting perpendicular to the reservoir's longitudinal axis which is often the case in natural settings. Of the three landslides types, sub aerial landslides will be considered, and the tests are performed by using solid blocks. Systematic variation of parameters like slide release height, slide volume, still water depth and upstream dam slope is made for each test. In detail, the impact of slide release height, slide volume and freeboard on the parameters defining the wave generation and dam overtopping has been investigated.

Furthermore, the statistical analysis of the instruments installed on the model setup to measure the overtopping volume and overtopping depth has been investigated.

1.4. Objectives

The main objective of this research is to undertake additional investigations on the existing physical scale model in order to gain more knowledge about the impact of impulse waves generated by sub aerial landslides. This knowledge should contribute to the process of developing a method to estimate parameters like overtopping volume and overtopping discharge.

The specific objectives of these thesis are:

- To see the relationship between slide volume and overtopping volume.
- Investigate the most dominant parameter that defines the slide impact velocity.
- Investigate the overtopping depth and overtopping volume pattern along the five dam crest sections of the present model setup.
- Define the relationship between the freeboard and overtopping volume.
- To compare the results of this study to literature.

1.5. Research questions

The research questions that will be answered at the end of this thesis are:

- What is the relationship between the slide volume, freeboard and overtopping volume?
- What is the dominant parameter that defines the slide impact velocity?
- For the specific model setup considered in this study, how is the overtopping depth pattern along the dam crest sections?
- For the specific model setup considered in this study, is the overtopping volume uniform along the dam crest sections?

1.6. Significance of the study

In Ethiopia, most of the dams are constructed in mountainous regions. However, landslide generated waves are not considered in the design of freeboard for new dams. Quantifying the amount of the overtopping due to landslide generated waves may be helpful in preventive measures and emergency preparedness. Hence, this study gives a basic insight to the process of developing a method to estimate the parameters like overtopping volume, in a way of defining the most dominant parameters to consider, due to landslide generated waves.

1.7. Outline of the thesis

This thesis is structured as follows. In chapter 1, Introduction on the landslide generated waves and in particular the history of the physical model where this research bases are introduced. Additionally, the scope, objectives, research questions and thesis outline of the study are presented. In Chapter 2, the three phases of slide impact on dams: wave generation, propagation and overtopping are discussed along with historical landslide across the world. Chapter 3 introduces the physical model where this research work bases. The basic parts and instruments used to measure parameters are presented. Chapter 4 discusses the experimental procedure followed for the whole tests. In addition, the statistical analysis of the instruments installed on the model setup is described based on different methods. Chapter 5 investigates the main results obtained from the experiments and compare the results to literature. Chapter 6 is conclusion and recommendation which summarizes the results from the present study and outline gaps for further research works.

2. Theoretical Background

2.1. Overview

The theoretical concepts about the landslide generated waves is discussed herein. The wave generation mechanism of landslide generated impulse waves depends on the initial position of the slide relative to the still water level (see Figure 4). These are (1) Subaerial slide, (2) Partially submerged slides, and (3) Submerged slides. The results described in this study are based principally on subaerial type of slide. For the alpine environment subaerial landslide generated impulse waves are particularly relevant because of steep valley sides, large slide masses and impact velocities, and a large number of artificial reservoirs. Subaerial types of landslides generate extremely high waves due to having high impact velocity, further leading to high amount of overtopping water over the dam. The wave generated by these slides is referred to a category of tsunami waves, where this study is concerned on.

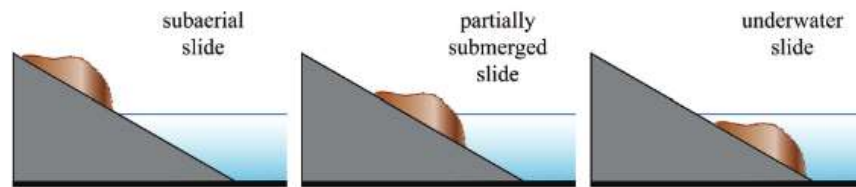


Figure 4: Wave generation mechanisms with subaerial, partially submerged, and underwater slide (Heller, 2007).

Heller (2007) presented the three main phases of the impulse wave phenomenon for subaerial slide as seen in Figure 5: (1) The wave generation zone or splash zone where there is a momentum transfer from landslide to water; (2) The wave propagation zone; and (3) The overtopping zone.

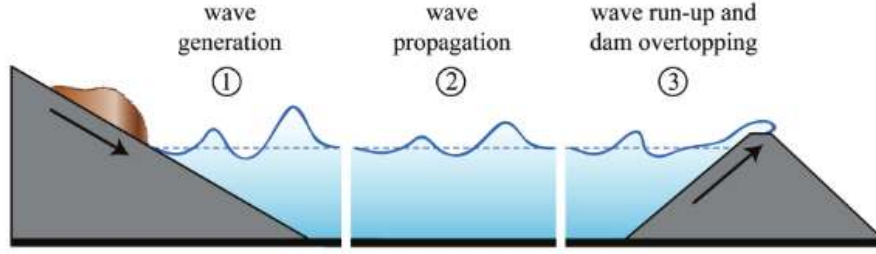


Figure 5: The three impulse wave phases (Heller, 2007).

Landslide generated impulse waves belong to the category of gravity waves, i.e. they are principally influenced by the gravitational force in contrast to capillary waves. They are different from perfect linear wave, which is consistent with the ideal sinusoidal wave. It is characterized by a relative height ($H/h < 0.03$) and by wave steepness ($H/L < 0.006$), where h is the still water depth, H is the wave height and L is the wave length. The wave height H is measured from the trough, i.e. from the lowest point on the wave surface to the crest, the highest point. The wave amplitude is the height from the undisturbed water surface to the wave crest. For a sign wave $a = H/2$. This no longer applies for impulse waves, which are generally nonlinear and vary from the perfect sine wave.

Further criterion that enables wave type description uses the ratio between wave amplitude and wave height (a/H) is presented by Zweifel et al. (2006) in three distinct non linearity levels as;

$$0.9 < a/H \leq 1 \text{ strongly nonlinear;}$$

$$0.6 < a/H \leq 0.9 \text{ moderately nonlinear;}$$

$$0.4 < a/H \leq 0.6 \text{ weakly nonlinear.}$$

Heller et al. (2009) allocated impulse waves to one of the following groups of waves: a) Stokes wave, b) cnoidal wave, c) solitary wave and d) bore.

a) Stokes wave

Stokes wave is a deep water to intermediate water wave ($L/h < 20$). The wave trough is somewhat flatter and longer than the wave peak. The wave particles do not move in a closed orbital fashion and, in consequence, slight transport of fluid mass takes place. Usually, wind generated waves belong to this category (Heller et al., 2009).

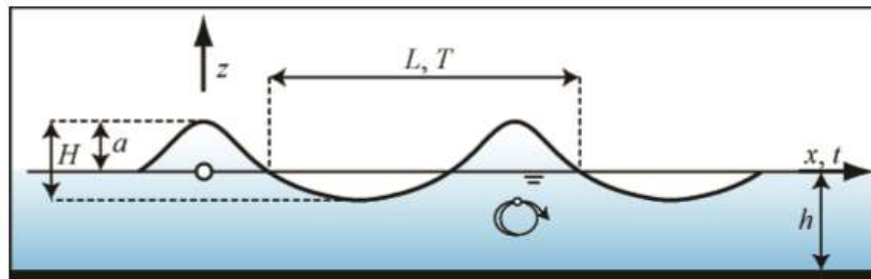


Figure 6: Characteristics of Stokes wave (Heller et al., 2009).

b) Cnoidal wave

Cnoidal wave is a periodic wave in intermediate or shallow water ($L/h > 2$). It has mainly an oscillatory character, but also exhibits open water particle orbits and hence transport of fluid mass. Using the equation for cnoidal waves, both the linear wave (sinusoidal wave) and the solitary wave are included as limiting cases (Heller et al., 2009).

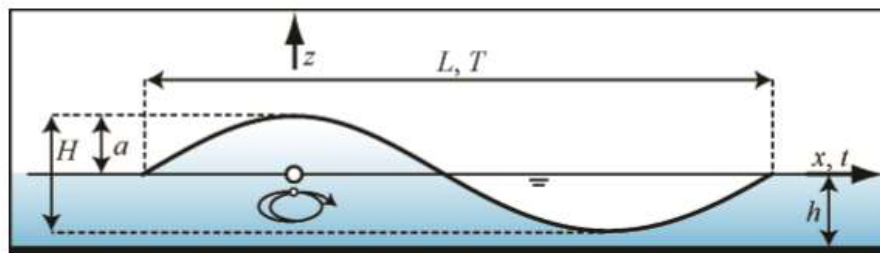


Figure 7: Characteristics of Cnoidal wave (Heller et al., 2009).

c) Solitary wave

Classic tsunamis, which are caused by the movement of tectonic plates, are often described with the solitary wave theory. It consists only of a wave peak but no trough. The wave amplitude is thus equal to the wave height $a = H$. In addition, the wave length $L = \infty$ and the wave is classed as a shallow-water wave ($L/h > 20$). Movement of the water particles is horizontal and as a consequence there is large fluid mass transport. The solitary wave theory is developed from the cnoidal wave theory, as the wave period $T \rightarrow \infty$ (Heller et al., 2009).

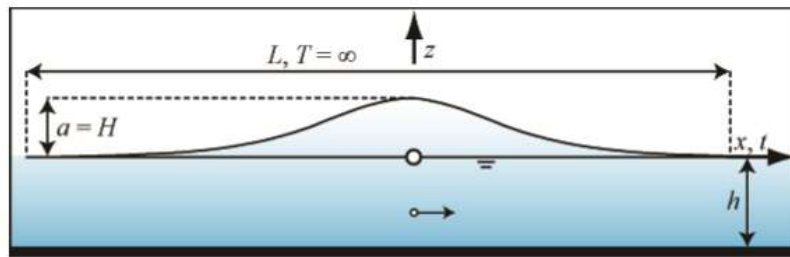


Figure 8: Characteristics of Solitary wave (Heller et al., 2009).

d) Bore

A bore is a shallow-water wave with horizontal particle movement which thus transports large fluid masses. Its profile is characterized by a steep front and a gently sloped back. Bore wave is generated when air is entrained at the crest or when the top of the crest curls over. It is a shallow water wave ($L/h > 20$) (Heller et al., 2009).

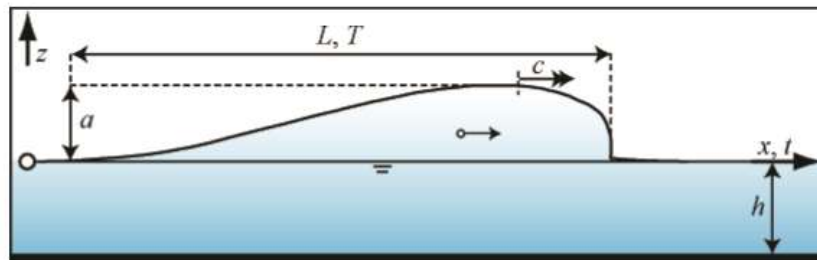


Figure 9: Characteristics Bore waves (Heller et al., 2009).

2.2. Wave generation and propagation

Slides falling into reservoir causes the transfer of kinetic energy from the slide mass to the water body creating impulse waves. These waves may overtop the dam, endangering the structural stability and causing downstream flooding. Hence, procedures for assessing the generation and propagation of landslide generated impulse waves form the integral part of an effective risk management strategy (Heller et al., 2009).

Studies on subaerial landslide generated impulse waves apply five methods (Heller et al., 2009). These are; (i) prototype specific model tests (Mueller D and Schurter M, 1993), (ii) numerical simulations (Gallati et al., 2005; Zhao et al., 2016; Zweifel et al., 2007) , (iii) empirical equations derived from field data (Ataie-Ashtiani and Malek-Mohammadi, 2007), (iv) analytical calculations (Di Risio and Sammarco, 2008), and (v) general applicable equations developed from model studies (Fritz, 2002; Fritz et al., 2003; Huber, 1982; Huber and Hager, 1997; Kamphuis and Bowering, 1970, 1970; Noda, 1970). A comparison is made between these methods based on the quality of the results, time requirement and cost, the clarity of the results and the efforts needed to determine the governing parameters required for each procedure. In landslide generated waves, generally applicable equations are quickly and easily used in practice. They can provide an initial estimate of the most important wave properties, such as wave height and runup height at the dam. This information on the effects of the impulse waves can help when taking decisions on any further preventive measures which may be needed.

Several general model studies were conducted in a wave channels (2D), where the waves are measured in two dimensions and propagate along the channel (Heller and Hager, 2011; Kamphuis and Bowering, 1970; Noda, 1970; Walder et al., 2003; Wiegel et al., 1970), or wave basins (3D), where the waves propagate freely and are measured in three dimensions (Panizzo et al., 2005). Among these studies, the experiments are based on block (Noda, 1970; Panizzo

et al., 2005; Walder et al., 2003; Wiegel et al., 1970) or granular slides (Fritz, 2002; Fritz et al., 2003; Heller et al., 2009; Huber, 1980; Zweifel et al., 2006) used for simulating the slides falling into the reservoir. Ataie-Ashtiani and Malek-Mohammadi (2007) presented results comprising of both approaches.

Although a considerable number of general model studies exist, the prediction of impulse wave features remains challenging. Most general model studies provide an empirical equation for the maximum wave features in the slide impact zone unlike the wave height, wave amplitude, wave period, and wave length at a specific location of the reservoir. However, defining these parameters at a specific location of the reservoir have to be available to determine the effects of impulse waves on dams (Heller et al., 2009). Accordingly, Heller Valentin and Hager Willi H. (2010) presented a complete empirical design criteria for both the maximum wave features in the slide impact zone and wave transformation in the wave propagation zone as a function of the impulse product parameter, P including the governing parameters in a wide test ranges. The seven governing parameters affecting the maximum wave height H_M , maximum wave amplitude a_M of period T_M , and length L_M are presented as (see Figure 10):

- h (m): still water depth;
- V_s (m/s): slide impact velocity;
- s (m): slide thickness;
- V_s (m³): bulk slide volume;
- ρ_s (kg/m³): bulk slide density;
- α (°): slide impact angle;
- d_g (m): grain diameter.

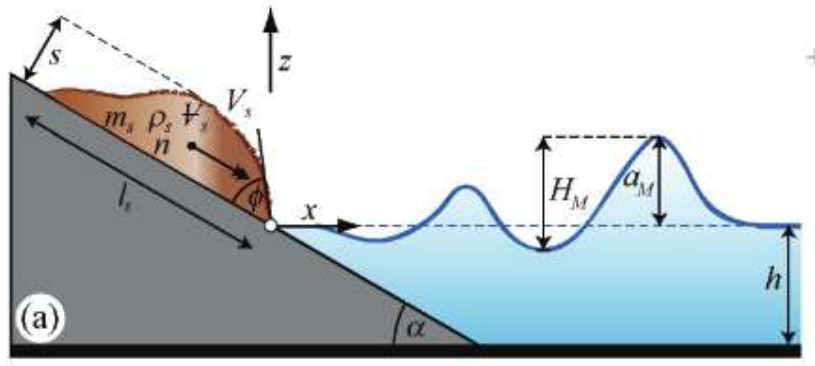


Figure 10: Sketch defining the governing parameters on impulse wave generation and the most important wave parameters (Heller et al., 2009).

The parameters like bulk slide bulk porosity n , slide mass m_s , slide length l_s , slide front angle Φ and time of underwater landslide motion t_s , defined as the time between slide impact and slide drop were not or only indirectly considered. For the data analysis of nearly all impulse wave parameters they have found one dimensionless parameter combination resulted in optimized coefficient of determinations, R^2 , impulse product parameter and described as;

$$P = FS^{1/2}M^{1/4}\{\cos[(6/7)\alpha]\}^{1/2} \text{ for } 0.17 \leq P \leq 8.13. \quad 1$$

where F is the slide Froude number, $F = V_s/(gh)^{1/2}$;

S is the relative slide thickness, $S = s/h$;

M is the relative slide mass, $M = m_s/(\rho_w b h^2)$.

For the calculation procedure of the impulse wave properties based on P , the limitations of the dimensional and non dimensional parameters listed in Table 1 should be applied.

Table 1: Limitations for the calculation of the impulse wave generation (Heller et al., 2009).

Term	Range	Meaning
Slide Froude number	$0.86 \leq F \leq 6.83$	$F = V_s / (gh)^{1/2}$
Relative slide thickness	$0.09 \leq S \leq 1.64$	$S = s/h$
Relative slide mass	$0.11 \leq M \leq 10.02$	$M = \rho_s V_s / (\rho_w b h^2)$
Relative slide density	$0.59 \leq D \leq 1.72$	$D = \rho_s / \rho_w$
Relative granulate density	$0.96 \leq \rho_g / \rho_w \leq 2.75$	ρ_g / ρ_w
Relative slide volume	$0.05 \leq V \leq 5.94$	$V = V_s / (b h^2)$
Bulk slide porosity	$30.7\% \leq n \leq 43.3\%$	n
Slide impact angle	$30^\circ \leq \alpha \leq 90^\circ$	α
Relative slide width	$0.74 \leq B \leq 3.33$	$B = b/h$
Relative radial distance	$5 \leq r/h \leq 30$	r/h
Wave propagation angle	$-90^\circ \leq \gamma \leq +90^\circ$	γ
Relative streamwise distance	$2.7 \leq X \leq 59.2$	$X = x/h$
Impulse product parameter	$0.17 \leq P \leq 8.13$	$P = FS^{1/2} M^{1/4} \{\cos[(6/7)\alpha]\}^{1/2}$

To study the reproducibility of waves generated by mesh packed slides compared with free granular slides, the maximum wave amplitude a_M and the maximum wave height H_M , as well as their decay along the propagation distance x were analyzed by (Heller and Hager, 2010). The relative maximum wave amplitude, $A_M = a_M/h$ and height $Y_M = H_M/h$ plus their decay $A(X)$ and $Y(X)$ along the relative distance measured from the location of the free water surface at the slide plane is presented as:

$$A_M = (4/9)P^{4/5} \quad 2$$

$$Y_M = (5/9)P^{4/5} \quad 3$$

$$A(X) = (3/5)(PX^{-1/3})^{4/5} \quad 4$$

$$Y(X) = (3/4)(PX^{-1/3})^{4/5} \quad 5$$

2.3. Slide movement type

Many classifications exist to distinguish and recognize different slide movement. A mass movement may be roughly defined by the material it consists, for example as landslide or rockfall. Cruden and Varnes (1996) identified five mass movement types as (see Figure 11);

- *Sliding*: the slide mass can move rotating along a curved concave slide plane or by translation on a plane. Slides can move as solid body or granular; in the latter case the bulk slide volume increases correspondingly as far as the impact zone.
- *Flowing*: the mass behaves as a viscous liquid.
- *Falling*: a fall may be considered to be mostly free fall, if the hill slope angle $> 76^\circ$; if this angle is smaller the mass falls onto the slope where it may break.

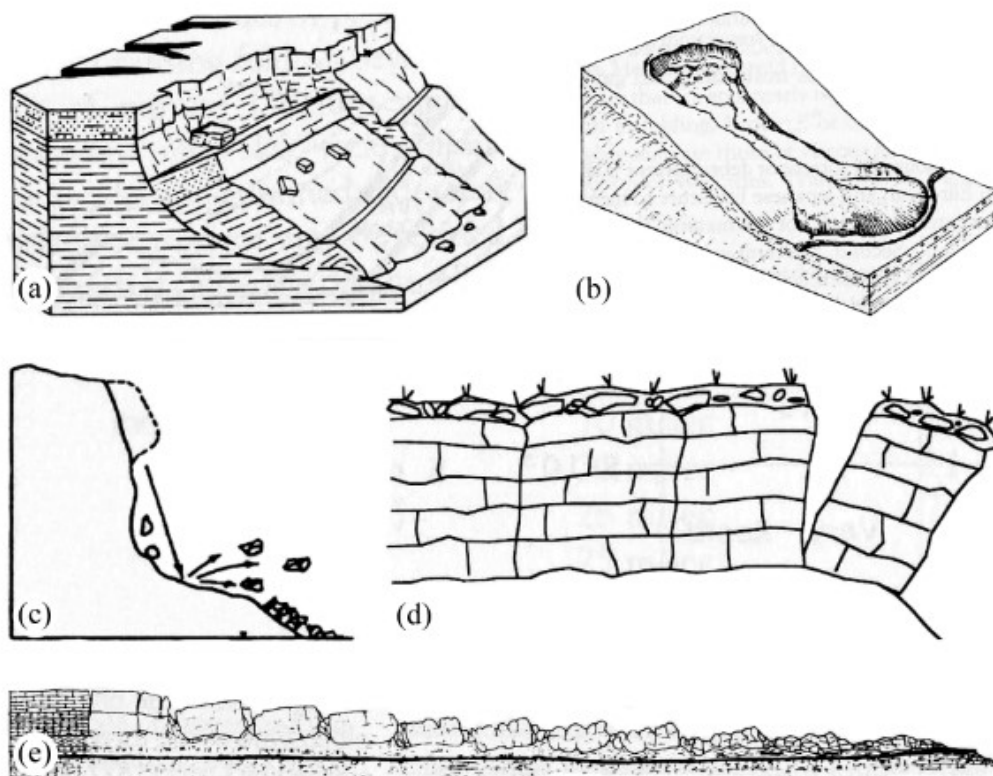


Figure 11: Mass movement types: (a) sliding, (b) flowing, (c) falling, (d) toppling, and (e) spreading (Cruden and Varnes, 1996).

- *Toppling*: the mass moves around a rotation point or axis located below its center of gravity.
- *Spreading*: the mass volume spreads and breaks into finer material. This movement type occurs when the slope angle is small (in the range between 0.3° to 5°) and is most often triggered by an earthquake.
- *Complex landslide*: the combination of two or more of the above landslide's type occur.

2.4. Wave runup and overtopping

Once a slide fall in to reservoir an impulse wave is generated, propagates and runs up towards the dam and it may overtop the dam if it exceeds the freeboard provided. The overtopping may cause flooding in downstream area and dam failure. This subsection gives an overview of the methods presented in the literature for defining parameters describing the overtopping process for example the run-up height and overtopping volume.

Figure 12 shows a sketch defining the governing parameters of impulse waves on dams. For a 2D case considered, the governing parameters considered are (Heller et al., 2009);

- H (m): wave height,
- a (m): wave amplitude,
- L (m): wave length,
- T (sec): wave period,
- h (m): still water depth,
- β ($^\circ$): dam face slope angle,
- f (m): freeboard,
- b_k (m): crest width.

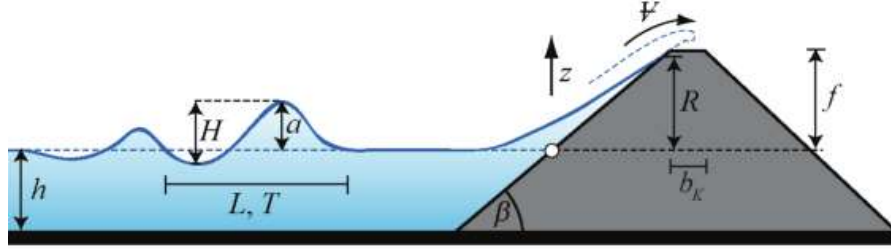


Figure 12: Sketch defining the governing parameters for the third phase, wave run up and dam overtopping (Heller et al., 2009).

Referring to Figure 12, one of the parameters defining the third phases of slide impact is the run-up height. There has been different formulas for defining the run up height in the literature (Hall and Watts, 1953; Müller, 1995; Synolakis, 1987). Among them, the formulas from Müller (1995) is discussed herein. The runup height, R can be determined by;

$$R = 1.25 \left(\frac{H}{h}\right)^{5/4} \left(\frac{H}{L}\right)^{-3/20} \left(\frac{90^\circ}{\beta}\right)^{1/5} h \quad 6$$

The following limitations applies:

Relative wave height	$0.011 \leq H/h \leq 0.521$
Wave steepness	$0.001 \leq H/L \leq 0.013$
Relative angle	$1.0 \leq 90^\circ/\beta \leq 4.9$.

From the engineering point of view, the overtopping volume is a basic parameter for hazard assessment during overtopping of dam due to landslide generated waves. Only a few studies conducted on landslide generated waves consider the dam overtopping. Among the studies conducted on landslide generated waves, only Müller (1995) and Kobel et al. (2017) provide equations to predict the overtopping volume. A discussion of the three step approach by Müller (1995) is given by Heller et al. (2009). The first step is to compute the runup height at an inclined plane with Eq. (4). Secondly, determine the overtopping volume per unit length dam

crest for freeboard $f = 0$, v_0 with Eq. (5). Lastly, the overtopping volume per unit length dam crest for $f > 0$, v is determined with Eq. (6).

The overtopping volume per unit length dam crest for freeboard $f = 0$, v_0 is calculated with;

$$v_0 = 1.45K \left(\frac{H}{h}\right)^{4/3} \left(\frac{T}{\sqrt{h/g}}\right)^{4/9} h^2 \quad 7$$

where $K [-]$ = Overfall coefficient for overtopping; $K = K_q K_b K_w^{3/2}$

$K_b [-]$ = Overfall coefficient for the crest width

$K_q [-]$ = Overfall coefficient for the steady case

$K_w [-]$ = Overfall coefficient for the increased flow energy compared with the steady case.

The equation to determine the overtopping volume per unit length dam crest for $f > 0$, v is

$$v = \left(1 - \frac{f}{R}\right)^{11/5} v_0 \quad 8$$

The approach presented by Müller (1995) for calculating the overtopping volume is a three step procedure where the runup height needs to be calculated first. Hence, Kobel et al. (2017) presented a one step formula for predicting the overtopping volume due to solitary waves on rigid dams considering two approaches. The first formula excludes the effect of upstream dam slope (Eq. (9)) while the other considers the effect of upstream dam slope (Eq. (10)).

$$\frac{v}{bh^2} = 1.42 \left[\varepsilon \left(\frac{h}{w}\right)^{2.5} \left(\frac{a_w}{b_k}\right)^{0.105} \right]^{0.8} = 1.42 W_1^{0.8}; \quad 9$$

with limitations; $0.35 < W_1^{0.8} < 0.95$, $0.07 < b_k/w < 0.53$.

$$\frac{v}{bh^2} = 1.35 \left[\varepsilon \left(\frac{h}{w}\right)^{(2/\varepsilon)(\beta/90^\circ)^{0.25}} \left(\frac{a_w}{b_k}\right)^{0.105} \right]^{0.7} = 1.35 W_2^{0.7}; \quad 10$$

with limitations; $0.15 < W_2^{0.7} < 0.95$, $0.07 < b_k/w < 0.53$.

Kobel et al. (2017) also presented two equations to estimate the maximum overtopping depth at the dam crest due to landslide generated waves. The normalized d_o/w versus the relative wave amplitude ε , the relative still water depth h/w , and the dam front face angle β follows the empirical fit with $R^2 = 0.95$ and $R^2 = 0.99$ respectively as:

$$\frac{d_o}{w} = 1.34 \left[\varepsilon \left(\frac{h}{w} \right)^{1.7} \left(\frac{\beta}{90^\circ} \right)^{0.25} \right] = 1.34E_1 ; \quad 0.40 < E_1 < 0.70 \quad 11$$

and,

$$\frac{d_o}{w} = 1.32 \left[\varepsilon \left(\frac{h}{w} \right)^4 \left[(\beta/90^\circ)^{-0.21} - \varepsilon \right] \left(\frac{\beta}{90^\circ} \right)^{0.16} \right] = 1.32E_2 ; \quad 0.10 < E_2 < 0.75 \quad 12$$

2.5. Historical impulse waves in reservoirs

Instabilities of bank slopes has been a challenging issue for the construction of dam projects, in the last decades the need for understanding this complex phenomenon has become of the utmost importance after several destructive incidents.

Historical landslide events with catastrophic consequences are readily available in literatures. One extreme event was Lituya Bay mega-tsunami in Alaska, 1958, triggered by an approximately 40 million yards of rock slide which caused the water to surge over the opposite wall of the inlet with maximum run up of 524 m. The rock slide falls from a maximum altitude of 914 m above sea level on average slope of 40° and it has dimensions of length from 730 m to 915 m, a maximum thickness of about 92 m and a center of gravity at around 610 m (Miller, 1960).

Another catastrophic event occurred on the 9th of October 1963 on the southern slope of the Vaiont dam. A mass of approximately 270 million m^3 collapsed into the reservoir generating a wave which crested 140 meters above the top of the dam and that still had a height of about 70

m downstream, at the confluence of the Vaiont with Piave valley (see Figure 13) (Genevois and Ghirotti, 2005).

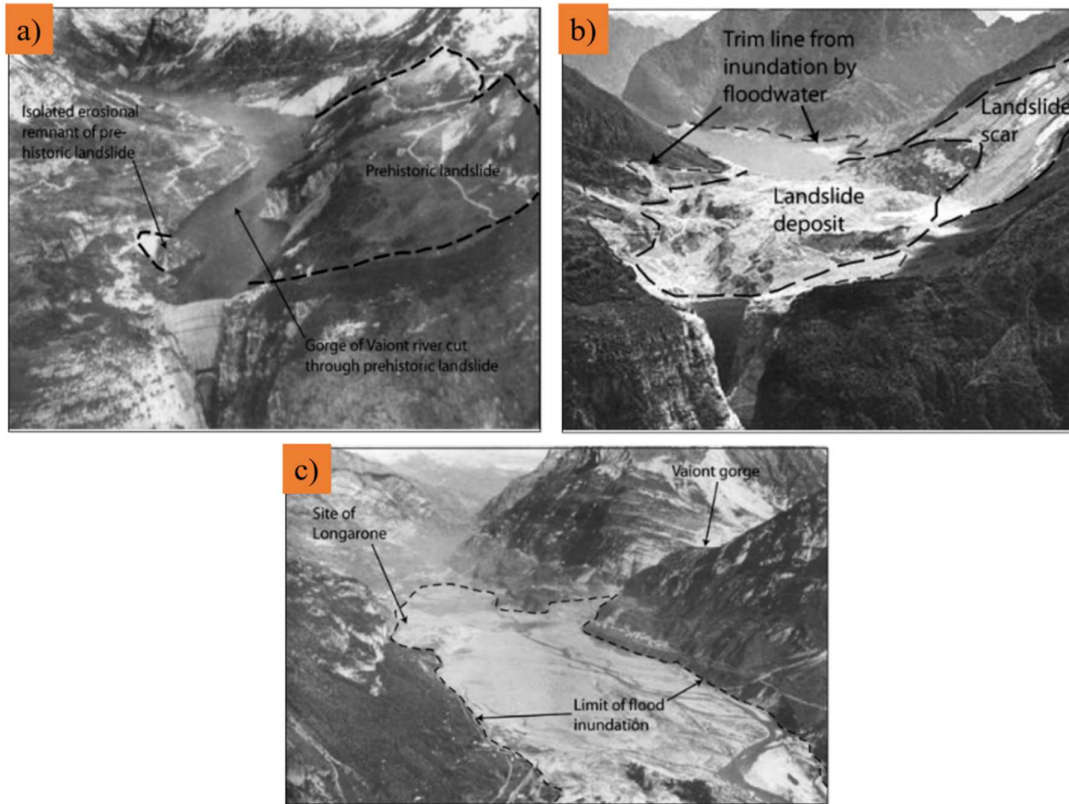


Figure 13: (a) Pre landslide view looking east over the dam in foreground, (b) Post slide view looking east, and (c) North looking view of the Piave valley after the flood (Steven and Simon, 2011).

After the tragedy, many studies and researches were carried out on the geological and geomechanical aspects and several attempts were made to explain the kinematics and dynamics of the landslide, also by using back analysis to study the many factors involved in the landslide development (Barla and Paronuzzi, 2013; Bosa and Petti, 2011; Manenti S. et al., 2016; Mueller, 1968; Steven and Simon, 2011).

Table 2 summarizes some of the deadly historical subaerial and partially submerged landslides generated waves along with their location, material of the slide, slid volume, slide ramp

inclination, still water depth, run up height and the number of people died due to the catastrophe.

Table 2: Some historical subaerial and partially submerged landslide generated impulse waves (Heller, 2007).

Year	Location	Material	W_s [$10^6 m^3$]	α [°]	h [m]	R [m]	Fatalities
1756	Tjelle (Norway)	Granite gneiss	15	>25	>200	46	38
1792	Shimbara (Japan)	Volcanic debris	500	10	64	10	>15000
1883	Krakatau (Indonesia)	Pyroxene/basalt	-	-	-	35	36000
1888	Ritter Island (Papua New Guinea)	Basalt/andesit	5000	10–15	1000	20	100
1905	Disenchantment Bay (Alaska)	Glacier ice	29	28	80	35	0
1934	Tafjord (Norway)	Gneiss	2–3	60	>200	62	41
1936	Loen (Norway)	Gneiss	1	25	<60	74	73
1958	Lituya Bay (Alaska)	Schist	31	40	122	524	2
1959	Pontesei reservoir (Italy)	Silt/clay debris	5	5	47	20	1
1963	Vajont reservoir (Italy)	Limestone	240	0–40	50	235.5	200
1971	Yanahuin Lake (Peru)	Limestone	0.1	45	38	30	400–600
1980	Mount St. Helens (USA)	Rock	430	-	-	200	0

3. Physical Model

Experiments have been conducted on a physical model in the hydraulic laboratory of NTNU. The model was built in 2010 as a reproduction of Geirangerfjord in Western Norway for studying the flood risk caused by the possible Åkneset rockslide. In the following years, this model has been rebuilt many times and a dam has been placed at one of its ends with the aim to understand better the impact of landslide generated waves on rockfill dams. For this reason, the model cannot be linked with an existing case, however, a scale of 1:190 has been applied as well to implement some technical procedures and analysis.

The model is composed of three main parts; the landslide, the reservoir and the dam which are described under experimental setup, Section 3.1. The sensors used for measuring different parameters like slide speed, wave height and overtopping depth are described in section 3.2, measuring instruments.



Figure 14: Picture of the physical model used for this study.

3.1. Experimental setup

The physical model in this study simulates a landslide falling perpendicular to reservoir's longitudinal axis, where an impulse wave is generated, propagates and runs up and overtops

the dam (see Figure 15). The main structural components of the model are: slide ramp where slide is placed, the basin simulating the reservoir and the dam which will be discuss herein.

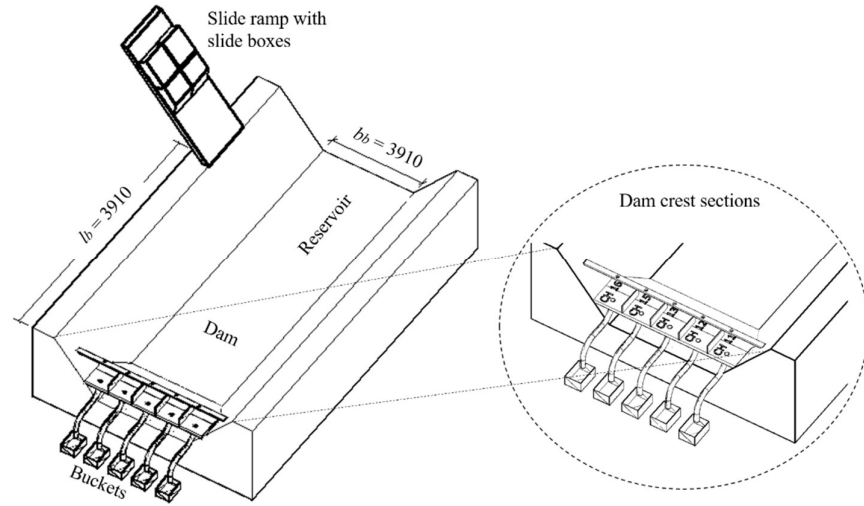


Figure 15: The 3D view of the model setup used for this study.

3.1.1. The slide

The landslide mechanism is composed of a 2 m long slide ramp inclined at an angle of $\alpha = 50^\circ$ where it is possible to place solid blocks of different size and arrangement (see Figure 15). The blocks are attached to each other with chains, the slide body is then attached to a steel panel with a hook. To measure the speed of the slide a speedometer is connected to the chains. When the hook is removed, the blocks slid into the reservoir generating impulsive waves.

The slide is simulated with rigid block (rectangular and parallelepiped blocks) as seen in Figure 16 with their dimension. Six different blocks are used which can be put in different arrangements.

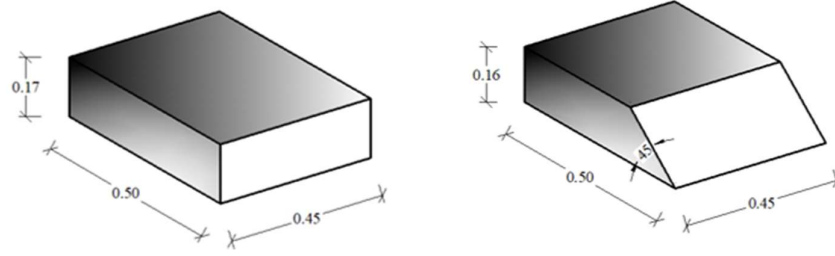


Figure 16: The rigid blocks used in this study parallelopiped block at the left and rectangular block at the right.

The blocks used in the experiments to simulate slide blocks were of a similar size (weight and volume) and form (rectangular) except block 7 and 8 which have a sloping edge (see Table 3)

Table 3: Slide block characteristics.

Block number	Weight (Kg)	Volume (m ³)	Shape
1	42.3	$3.83 \cdot 10^{-2}$	Rectangular parallelepiped
2	42.4	$3.83 \cdot 10^{-2}$	Rectangular parallelepiped
3	42.3	$3.83 \cdot 10^{-2}$	Rectangular parallelepiped
4	42.4	$3.83 \cdot 10^{-2}$	Rectangular parallelepiped
7	37.7	$3.06 \cdot 10^{-2}$	Right trapezoid solid
8	37.6	$3.06 \cdot 10^{-2}$	Right trapezoid solid

For each model setup, the landslide is composed of blocks which are arranged in different arrangements. Hence, four different block configurations are used: two blocks aligned horizontally (2H) and vertically (2V), four blocks (4 Blocks) and six blocks (6 blocks). In each block arrangement the slide blocks are tapered at an angle of $\Phi = 45^\circ$ at the front to have the highest impact (Sælevik et al., 2009) (see Figure 17). A constant value of slide front angle is taken because its effect is found negligible in (Heller, 2007).

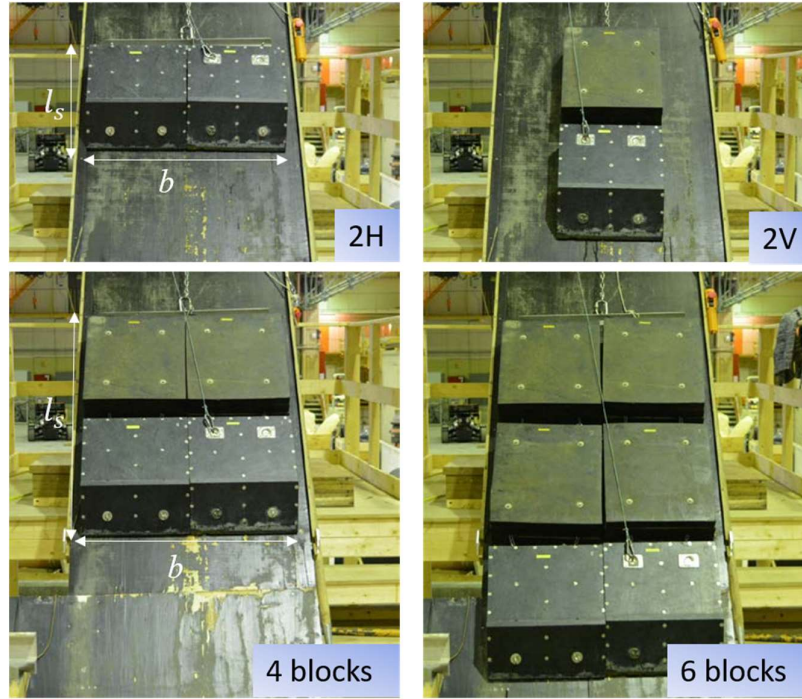


Figure 17: Picture showing different block configurations used in this study.

3.1.2. The reservoir

The reservoir was simulated with a trapezoidal basin having planar slope sidewalls of water-resistant plywood covered with a concrete paste for increasing the roughness. Unlike previous studies conducted in a rectangular prismatic channel, the slide is released perpendicularly to the reservoir's longitudinal axis (see Figure 15). The right side of reservoir basin is extended to the slide ramp where the slide is released (Figure 18).

The basin has 4.5 m length, 1.7 m width at the bottom, 2.2 m width at dam crest level, with a total reservoir capacity of 2.5 m³ corresponding to 860 m length, 320 m width at the bottom and 17 mill.m³ in prototype. Once the slide is released, the wave is generated and propagates along the reservoir. Hence, to measure the wave properties, like the wave amplitude with time, nine wave gauges are installed along the reservoir which will be discussed in Section 3.2.



Figure 18: Picture of the reservoir in the laboratory.

Among the dominant parameters which influences the overtopping volume is freeboard: the vertical distance between the water surface and dam crest. Considering a risk based philosophy, dams are categorized by estimating the consequence from the dam failure and freeboard is one of the required parameters that defines the consequence class. In this study, freeboard value related to class 3 ($f = 4.5$ m) and class 4 ($f = 6$ m) dams in Norway respectively are selected. For a constant dam height $H = 0.32$ m, $f = 0.024$ m and $f = 0.032$ which relates to a still water depth of $h = 0.296$ m and $h = 0.032$ m respectively are used throughout the tests.

3.1.3. The dam

Dams having an upstream slope of 1: 1.5 and 1: 2.25 are used for the analysis (Figure 19 a and b respectively). To clearly see the distribution of the overtopping waves along the dam crest, the dam crest is divided into five different sections, referred to in the following as dam crest sections as seen in Figure 20.

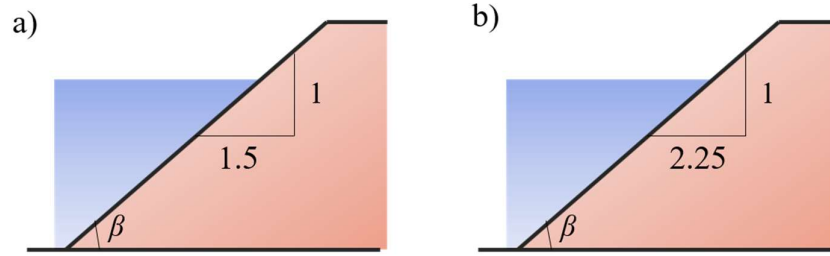


Figure 19: Section view of the dam.

The overtopping depth with time over the dam crest is measured by the five ultrasonic sensors placed in each dam crest sections (Figure 20).

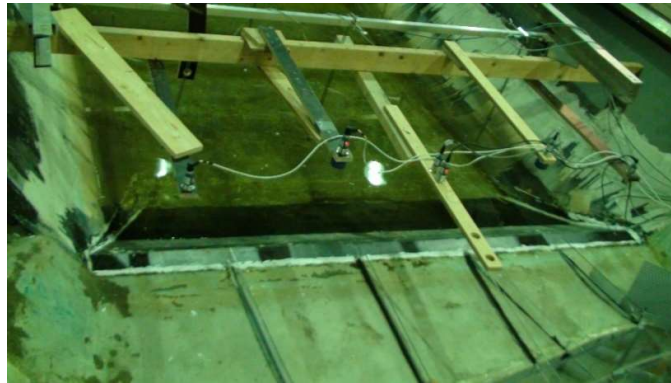


Figure 20: Picture of the dam crest divided into five sections.



Figure 21: Picture of the five buckets collecting the overtopping water over the five dam crest sections.

The water that overtops the dam crest sections are guided to the corresponding buckets with 100 mm diameter pipes as seen in Figure 21. The five collecting buckets have the same size of 685 mm x 465 mm x 315 mm.

3.2. Measuring Instruments

Different sensors are used in the model to measure the landslide speed, the wave height, the overtopping depth and overtopping volume. Table 4 illustrates those measuring instruments with their number and their use in the model setup.

Table 4: Types of sensors used for the study

No.	Type of sensor	Number	Sensor designation	Measures
1	Wave gages	9	CH1, CH2,CH3,CH4,CH5,CH6,CH7,CH8,CH9	Wave height
2	Ultrasonic sensor	5	CH11,CH12,CH13,CH15,CH16	Overtopping depth
3	Ultrasonic sensor	1	-	Overtopping volume
4	Rotational sensor	1	CH10	Slide impact velocity

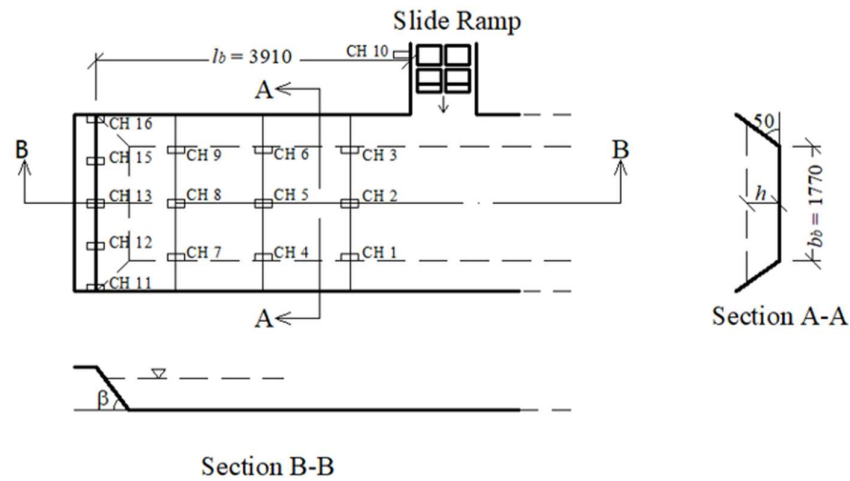


Figure 22: Placement of all the sensors used in this study.

Referring to Figure 22 nine wave gauges (CH 1 to CH 9) of type ‘DHI wave meter 102E’ are installed to measure the wave height in the reservoir (Figure 22). The wave gauge is composed

of two parallel electrodes inserted partially in the water and works based on the electrical conductivity principle in fluid. These sensors need to be calibrated before each test, every time the still water level changes in the reservoir, in order to get more accurate results. The calibration procedure is described in section 3.3.

Five ultrasonic sensors (CH 11 to CH 16) are placed over the dam crest sections to measure the overtopping depth with time (see Figure 23). The working principle of the ultrasonic sensor is that, it emits a high frequency sound pulse and evaluates the voltage measurement considering the time taken for the signal to be reflected back. The calibration procedure for the ultrasonic sensor is described in section 3.3.



Figure 23: The picture of five ultrasonic sensors for measuring the overtopping depth along the five dam crest sections.

Another sixth ultrasonic sensor is used to measure the overtopping volume over the dam crest which is collected in the five buckets (see Figure 24). The measurement is taken manually from each bucket. A reading is taken before the start of the test and then after the overtopping occurs, hence the overtopping volume is the difference between the two values. In order to change the distance, read by the ultrasonic sensor to volume, a calibration factor is needed, which is discussed in section 3.3.



Figure 24: Ultrasonic sensor used for measuring the overtopping volume over the dam crest in each bucket.

A rotational sensor (CH 10) is located at the slide ramp (Figure 22). It is connected to the rigid blocks through the hook and composed of a rope and volt meter (Figure 25).

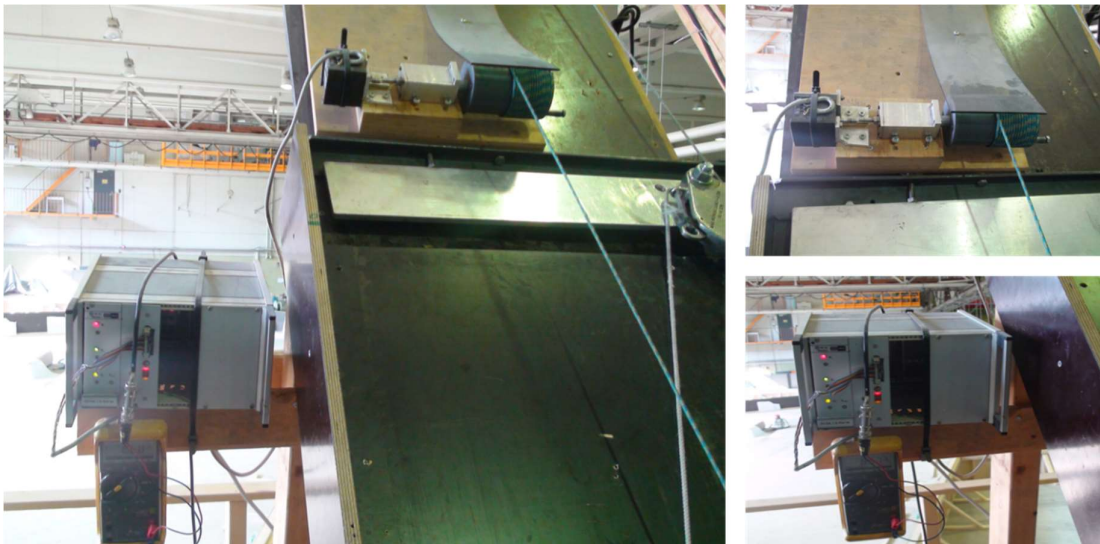


Figure 25: Picture of rotational sensor for measuring the slide speed.

A triggering wheel is placed at the side of the ramp connected with the slide blocks, and to send a signal when the sliding process starts. When the slide is released by disconnecting the hook, the slide moves down the slide ramp, meanwhile the rope unrolls together with it. Then, the distance covered by the slide with time is recorded with a voltmeter and sent to the computer so that it is possible to extract the speed of the slide. However, the rotational sensor needs to be calibrated to have a correlation between the voltage reading and the distance covered by the slide, which is discussed later in section 3.3.

3.3. Instrument calibration

The nine wave gauge sensors (CH 1 to CH 9) need to be calibrated before each test, since every time an experiment is done, still water level changes in the reservoir.

The calibration procedure is as follows;

- Fill up the reservoir to the desired level using the piezometer.
- After filling up the reservoir and the water being still, set each of the channels to 0 V.
- Elevate the wave gauges by adding 50 mm steel bars and then set it to – 1V. The same applies by lowering the wave gauges in 50 mm and setting to + 1V.
- When the calibration is over, the wave gauges are put in the initial position (zero level).
- Through these procedures a calibration factor of 50 mm for 1 V is obtained.

The five ultrasonic sensors (CH 11 to CH16) were calibrated for each dam by moving a block of 50 mm steel block under it and read the voltage reading. Then the voltage reading between the measurements with and without the steel block is used as a calibration factor. Hence, a calibration factor for 50 mm reading is described for each sensor as;

CH11	CH12	CH13	CH15	CH16
3.37	3.86	2.31	3.67	2.03

The sixth ultrasonic sensor used to measure the overtopping water collected in the five buckets. It is calibrated by adding 1 l (until 8 l) of water in to the bucket and measure the reading from the sensor. Repeat this step many times so that we get consistent result. This way a calibration factor of 1 l for -5.5 mm is found in many trials.

The rotational sensor (CH10) is calibrated by pulling out the rope for example with 0.5 m, 1 m, 1.5 m and 2 m length and record the voltage reading on the voltmeter. Then the slope of the graph for the distance and the voltage reading is taken as a calibration factor. Then by repeating this procedure many times average calibration factor of 1 m to -1.33 is obtained.

All the data collected from the ultrasonic sensors, wave gauges and rotational sensor is collected in '.csv' format and transferred to a computer and then processed with '*Agilent software*' (see Figure 26). The sampling rate of the software is 200 samples per second.

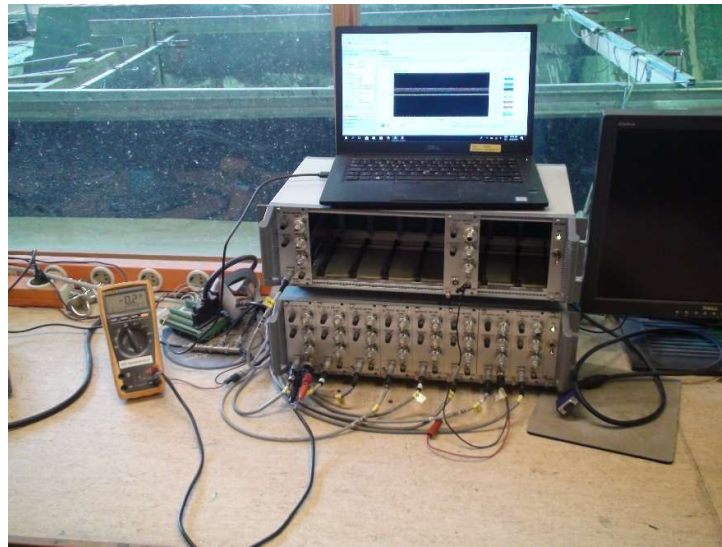


Figure 26: A computer with Agilent software.

3.4. Froude's Model law

Laws of similitude is derived by looking at the relation between different forces, such as inertial, gravitational, viscous and surface tension. Depending on which forces are the most dominant in the process, different scaling criterion are applied as:

- Froude similarity: is used for gravity driven flows where inertial and gravity forces dominate, typically applied for models of rivers and hydraulic structures.
- Reynolds similarity: is used for flows with high internal friction where inertial and viscous forces are significant.
- Euler similarity: is used for flows through turbines and pumps where the pressure forces are the most important
- Weber similarity: is used for aerated flows where surface tension forces are significant

The perfect similitude between a hydraulic model and its prototype requires geometric, kinematic, and dynamic similarity (Novák, 1981; Yalin, 1971). For geometrical similarity, all ratios of the corresponding linear dimensions (e.g. length) must be identical between the model and the prototype; the kinematic similarity applies to all components of the vectorial motions for all particles at any time (e.g. time) involved in the flow process; in addition, the ratios of all vectorial forces (e.g. gravity) in the two systems must be identical for dynamic similarity (see Table 5).

Table 5: Similarity types between a model and a prototype. The subscripts, m and p , stand for model and prototype.

Similarity	Notation	Ratio
Geometric similarity	L_r	L_m/L_p
Kinematic similarity	V_r	V_m/V_p
Dynamic similarity	F_r	F_m/F_p

The relevant forces of landslide generated impulse waves are the inertial, the gravitational, the viscous, the surface tension, and the elastic compression forces (Table 6). No fluid may satisfy all force ratio requirements if the model is different from the prototype (Hughes 1993; Heller 2007). The relevant force ratio for impulse wave generation is the square root of the inertial to the gravity forces, i.e. the Froude number. According to the Froude similitude the slide Froude number in the model and in its prototype have to be identical (Eq. (13)).

$$F_r = \frac{F_m}{F_p} = 1 \quad 13$$

Table 6: list of forces for landslide generated impulse waves.

Symbol	Force	Formula	Unit
F_i	Inertial force	$\rho L^2 V^2$	Newton [N]
F_g	Gravitational acceleration	$\rho L^3 g$	Newton [N]
F_μ	Viscous force	$\mu V / L$	Newton [N]
F_σ	Surface tension	σL	Newton [N]
F_e	elastic compression	$E L^2$	Newton [N]
F_{pr}	Pressure force	ρL^2	Newton [N]

Table 7: The most common scale ratios for scaled model with Froude's model law.

L=scale factor	Geometric similarity	Conversion factor for common scale ratios		
		1:25	1:50	1:100
GEOMETRY				
Length	L_r	25	50	10^2
Area	L_r^2	625	2500	10^4
Volume	L_r^3	15.625	125 000	10^6
KINEMATICS				
Time	$L_r^{1/2}$	5	7,07	10
Velocity	$L_r^{1/2}$	5	7,07	10
Acceleration	1	1	1	1
Discharge	$L_r^{5/2}$	3 125	17 675	10^5
DYNAMICS				
Mass	$(L^3\rho)_r$	15 625	125 000	10^6
Force	$(L^3\rho)_r$	15 625	125 000	10^6
Pressure	$(L\rho)_r$	25	50	10^2
Impulse	$(L^{7/2}\rho)_r$	78 125	883 750	10^7
Energy	$(L^4\rho)_r$	390 625	6 250 000	10^8
Effect	$(L^{7/2}\rho)_r$	78 125	883 750	10^7

The Froude similitude considers only the inertial and gravity forces whereas the fluid viscosity, the surface tension, and the elastic compression forces are neglected. They may cause scale effects if their effects are large. According to Le Mehaute (1976), the Froude similitude applies to free surface flows with negligible viscous forces e.g. prior to wave breaking or to short, highly turbulent flows such as in the wave breaking zone. The impulse wave generation is such a highly turbulent flow. In the present study gravity is the prevalent force, since it is free surface flow and as such it is Froude's model law which is in effect.

Table 7 gives the most common scale ratios for scaled models based on Froude's model law (Lysne et al.).

4. Test Procedure and Data Reliability

This section describes all the phases that compose the experimental procedure and the main operations involved. The statistical analysis of the sensors is also explained here.

4.1. Experimental procedure

Before conducting each test, the reservoir is first filled up by a pump to the desired water level. The water level through the reservoir is monitored by the piezometer (Figure 27). After the water being set, calibration of the wave gauges is done (CH1 to 9), a calibration procedure is discussed in section 3.3. Meanwhile, the slide blocks are placed on the slide ramp for a specific slide release height. Then, calibration of rotational sensor (refer to section 3.3) is conducted by unrolling the rope for different lengths. Before releasing the slide, make sure that the start trigger is in contact with the slide blocks, otherwise the distance covered by the slide will not be recorded.



Figure 27: Picture of a Piezometer used for measuring the reservoir water level.

The next step is to turn on the cameras to record the overall process and the ‘*Agilent software*’ so that the data from all the measuring sensors is collected. This data is collected in a computer with ‘.csv’ format and exported to ‘.xls’ for data analysis except the overtopping volume which is measured manually with ultrasonic sensor. The slide can be released by pulling the rope which unhooks the slide from its initial position.

The final step is to lift the blocks felt to the reservoir and get ready for another test with different parameters. Meanwhile, the still water level needs to be refilled with the pump since some of it is lost by overtopping over the dam crest.

Table 8: Summary of performed experiments in this study.

U/s dam Slope	Freeboard, f	Block arrangement	Release Height (h_o) (m)	N° of test
1: 1.5	4.5	2H	2	3
			1.5	3
			1	3
1: 1.5	4.5	2V	2	3
			1.5	3
			0.5	3
1: 1.5	4.5	4	1.5	3
			1	3
			0.5	3
1: 1.5	4.5	6	0.5	3
			1	3
1: 1.5	6	6	1	3
			0.5	3
1: 1.5	6	4	1.5	3
			1	3
			0.5	3
1: 1.5	6	2H	2	3
			1.5	3
			1	3
1: 1.5	6	2V	2	3
			1.5	3
			0.5	3
1: 2.25	4.5	2H	2	3
			1.5	3
			1	3
1: 2.25	4.5	2V	2	3
			1.5	3

			0.5	3
1: 2.25	4.5	4	1.5	3
			1	3
			0.5	3
1: 2.25	4.5	6	0.5	3
			1	3
1: 2.25	6	6	1	3
			0.5	3
1: 2.25	6	4	1.5	3
			1	3
			0.5	3
1: 2.25	6	2H	2	3
			1.5	3
			1	3
1: 2.25	6	2V	2	3
			1.5	3
			0.5	3

44 experiments were carried out in this study. Each test is repeated three times for data reliability, hence a total of 132 test were conducted (see Table 8). The average of the three repeated test results is considered in the analysis (the parameters like the slide speed, the wave height and overtopping volume).

4.2. Data reliability

In experimental study the reliability and trend of the data should be analyzed before proceeding to the data analysis. This has been done by repeating the same test for 20 times and estimating the repeatability of the results obtained. Hence, the result from the experiment for the same model setup (Test no. 1.5s_4.5_2H_150) is used for testing the instrument reliability.

The normal distribution is the most important probability distribution in statistics because it fits many natural phenomena for example, physical measurements often follow a normal distribution. The normal distribution is a probability function that describes how the values of a variable are distributed. It is a symmetric distribution where most of the observations cluster around the central peak and the probabilities for values further away from the mean taper off equally in both directions. For each mean and standard deviation of a theoretical data, normal

distribution can be determined. Then, this theoretical normal distribution can be compared to the actual distribution of the data. For assessing whether the data are normally distributed or not, different methods are available in the literature. They can be categorized in to two broad categories: graphical and statistical. In this study, among the graphical (Q-Q probability plots and Cumulative frequency (P-P) plots) and statistical (Kolmogorov- Smirnov test and box plot) are applied in the next section. This method has been applied to the experimental results like maximum wave height recorded with wave gauges (CH1 to CH9), overtopping depth recorded with the ultrasonic sensors (CH11 to CH16) and overtopping volume measured in bucket 1 through 5.

4.2.1. Normal distribution by Graph

The experimental data distribution about the mean is illustrated by graph here in. Hence, a plot is made between a normal distribution scatter and histogram for frequency bins of raw data for the maximum wave height, maximum overtopping depth and the total overtopping volume (see Figure 28). This procedure has been applied to all the 20-sample data. Based on this approach, the data is likely to be normally distributed if the shape of the histogram resembles with a bell curve. Figure 28 illustrates some plots of the overlapping between the histograms and the normal distribution curve for the wave height (CH5), overtopping height (CH12) and total overtopping volume. Based on the plots, the histograms seem to resemble a normal distribution. However, the normal distribution graphs of CH1, CH4 and CH16 doesn't seem to resemble on histogram. Hence, further analysis is needed for these channels.

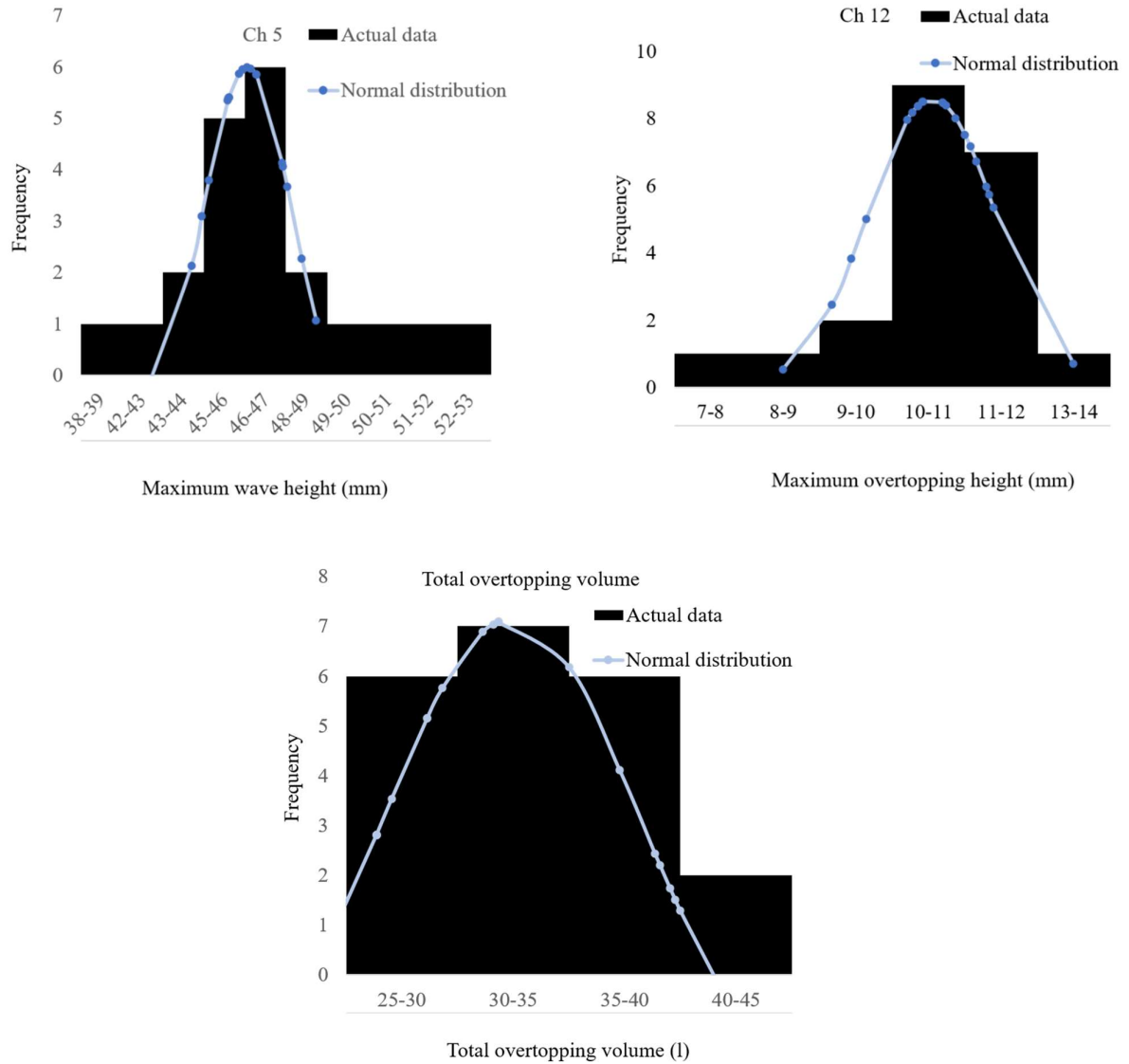


Figure 28: Plot between a histogram and normal distribution curve for CH5, CH12 and the total overtopping volume in the buckets.

4.2.2. Kolmogorov-Smirnov test for Normality

The Kolmogorov-Smirnov goodness of fit test is further applied to all the experimental data in order to verify whether the data actually follows Gaussian distribution. It quantifies the distance between the empirical distribution function of the sample (F), evaluated as cumulative relative frequency, and the cumulative normal distribution function, S_n .

Let $x_1, \dots, x_n$ be an ordered sample with $x_1 \leq \dots \leq x_n$ and define $S_n(x)$ as follows:

$$S_n(x) = \begin{cases} 0, & x < x_1 \\ k/n, & x_k \leq x < x_{k+1} \\ 1, & x \geq x_n \end{cases}$$

where n is the number of observations.

Now suppose that the sample comes from a population with cumulative distribution function $F(x)$ and define D_n as follows:

$$D_n = \max_x |F(x) - S_n(x)|$$

If $D_{n,\alpha}$ is the critical value from the table, then $P(D_n \leq D_{n,\alpha}) = 1 - \alpha$. D_n can be used to test the hypothesis that a random sample came from a population with a specific distribution function $F(x)$. If

$$\max_x |F(x) - S_n(x)| \leq D_{n,\alpha}$$

then the sample data is a good fit with $F(x)$.

The experimental data is further analyzed by Kolmogorov-Smirnov data test (KS test) with significant level of 0.01, 0.05, 0.1, 0.15 and 0.2.

Table 9: Parameters for normal distribution with KS test considering wave heights.

	CH1	CH2	CH3	CH4	CH5	CH6	CH7	CH8	CH9
D_n	0.335	0.425	0.099	0.125	0.134	0.117	0.123	0.087	0.210
α	—	—	0.2	0.2	0.2	0.2	0.2	0.2	0.2
$D_{n,\alpha}$			0.21	0.21	0.21	0.21	0.21	0.21	0.21

Table 10: Parameters for normal distribution with KS test considering the maximum overtopping depth.

	CH11	CH12	CH13	CH15	CH16
D_n	0.21	0.12	0.10	0.15	0.37
α	0.2	0.2	0.2	0.2	—
$D_{n,\alpha}$	0.21	0.21	0.21	0.21	0.21

Table 11: Parameters for normal distribution with KS test considering the total overtopping volume.

	Bucket 1	Bucket 2	Bucket 3	Bucket 4	Bucket 5
D_n	0.15	0.22	0.17	0.15	0.19
α	0.2	0.2	0.2	0.2	0.2
$D_{n,\alpha}$	0.21	0.21	0.21	0.21	0.21

As we can see from the above tables, $D_n < D_{n,\alpha}$ for all values except for CH1, CH2 and CH16. We conclude that the data is a good fit with the normal distribution except for those channels. Hence, further analysis is made for these channels; CH1, CH2 and CH16.

4.2.3. Q-Q Plots (Quantile-Quantile plot)

A Q-Q plot is a scatterplot created by plotting two sets of quantiles against one another. If both sets of quantiles came from the same distribution, we should see the points forming a line that is roughly straight. Q-Q plots display the observed values against normally distributed data which is represented by line. It is a graphical tool used to assess if a set of data came from theoretical distribution such as Normal or exponential.

When using a QQ plot to see whether a data is normally distributed or not, we can create a scatter diagram between range R_1 consisting of the elements $X_1, \dots, X_n$ in ascending order and R_2 consisting of the values NORM.INV (1/2n, $\bar{x}$, s), ..., NORM.INV (1-1/2n, $\bar{x}$, s), where $\bar{x}$ =

AVERAGE (R_1) and $s = \text{STDEV} (R_1)$. Alternatively, you can create a scatter diagram between range R_1 consisting of the standardized elements $Z_1, \dots, Z_n$, where each $Z_i = \text{STANDARDIZE} (x_i, \bar{x}, s)$, and R_2 consisting of the values $\text{NORM.S.INV}(1/2n), \dots, \text{NORM.S.INV}(1-1/2n)$.

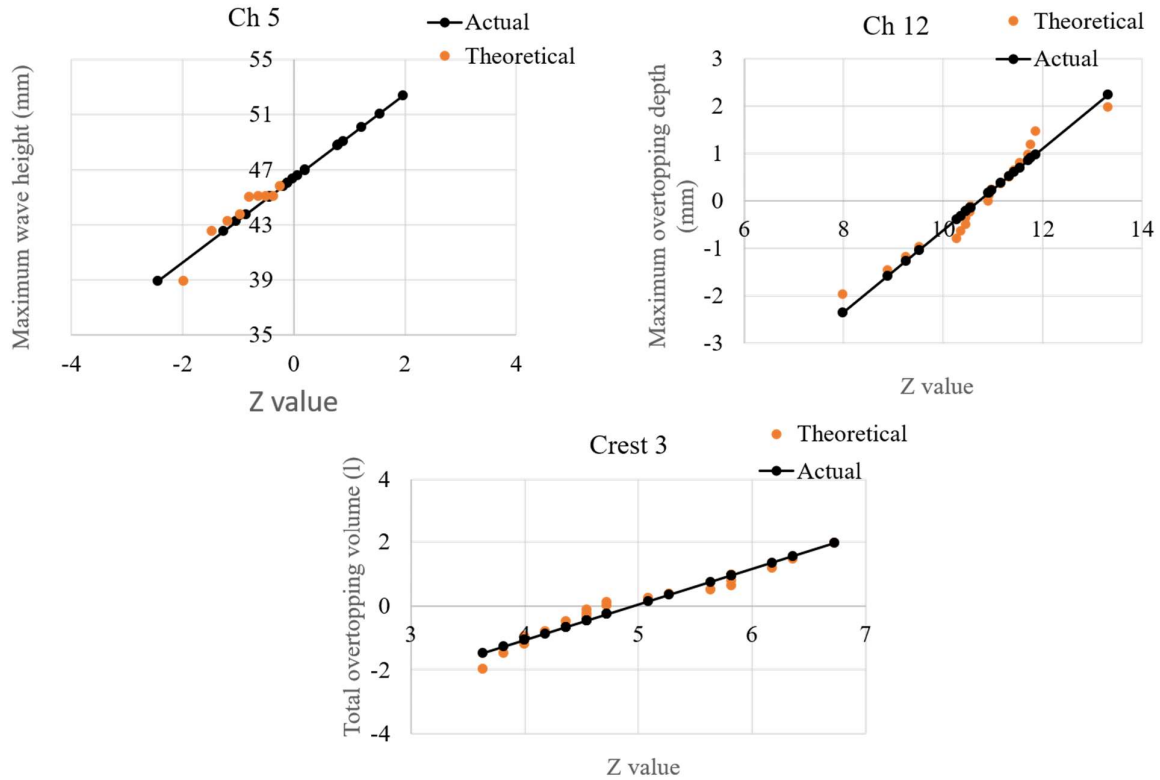


Figure 29: QQ plot between the theoretical and actual sample data for CH5, CH12 and crest 3.

Sample QQ plots for wave height (CH5), overtopping depth (CH12) and the overtopping volume (crest 3) is shown in Figure 29. If the points on the scatter plot align with the diagonal line $y = x$ then the data set conforms to the distribution. Hence, the plots between actual and theoretical sample data seem to overlay except from the output CH1, CH2, CH4 and CH16.

4.2.4. Box plots

A box diagram (box and whisker chart) shows the distribution of data in quartiles (Q1, Q2, Q3) and highlighting average and outliers, where Q1, Q2 and Q3 are the 25th, 50th and 75th percentile of the sample data respectively (see Table 12).

Table 12: Parameters for box plot in investigating the outliers considering the maximum wave height.

	CH1	CH2	CH3	CH4	CH5	CH6	CH7	CH8	CH9
Minimum	78.11	52.19	44.10	2.11	38.94	40.25	29.45	25.36	25.64
Q1	82.54	79.06	47.64	42.79	45.08	44.33	32.66	32.23	29.72
Median	83.82	89.01	50.34	61.96	46.05	47.15	33.94	34.50	32.98
Q3	87.41	93.06	53.12	84.40	48.74	48.70	35.02	36.99	36.20
Maximum	191.76	100.00	60.30	92.69	52.34	51.88	40.43	42.49	56.20
Mean	91.90	84.14	51.16	59.89	46.36	46.27	33.93	34.10	35.28
Range	113.65	47.81	16.20	90.58	13.40	11.63	10.98	17.13	30.56
IQR	4.87	13.99	5.48	41.61	3.66	4.37	2.36	4.76	6.48
Q1-IQRx1.5	75.23	58.07	39.41	-19.62	39.60	37.77	29.13	25.09	19.99
Q3+IQRx1.5	94.72	114.05	61.34	146.81	54.22	55.26	38.56	44.13	45.93

Box plots has been made for the maximum wave height (Figure 30), the overtopping depth (Figure 31) and the total overtopping volume (Figure 32). In these figures, the dots outside of the box shows outliers which are not included in Inter quantile range i.e. between upper and lower limit. After removal of outliers, CH1, CH2 and CH16 have passed KS test with a significance level of 0.1 and 0.2, 0.01 respectively. And the graph between the theoretical and actual data values seem to fit. Figure 33 shows the plot between the theoretical and actual sample data value before and after removal of outliers of CH1, CH2 and CH16. The plots for these channels indicate that after the removal of the outliers, the graphs of the theoretical and actual sample data values for all the three channels seem to rely on each other.

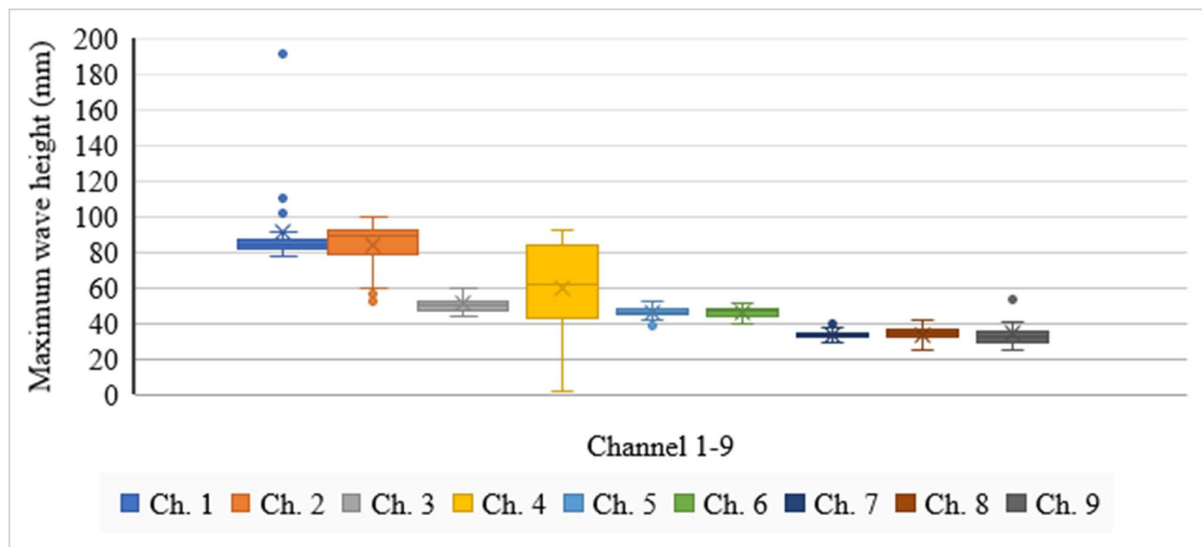


Figure 30: Box plot for the maximum wave height recorded with CH1 to CH9.

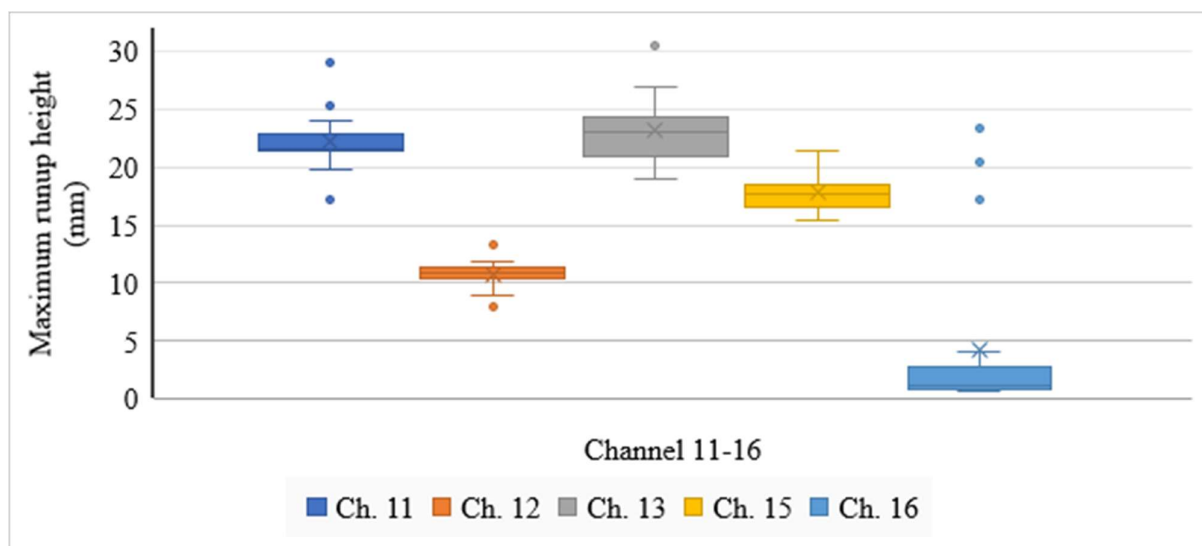


Figure 31: Box plot showing the outliers for maximum overtopping depth recorded with CH11 to CH16.

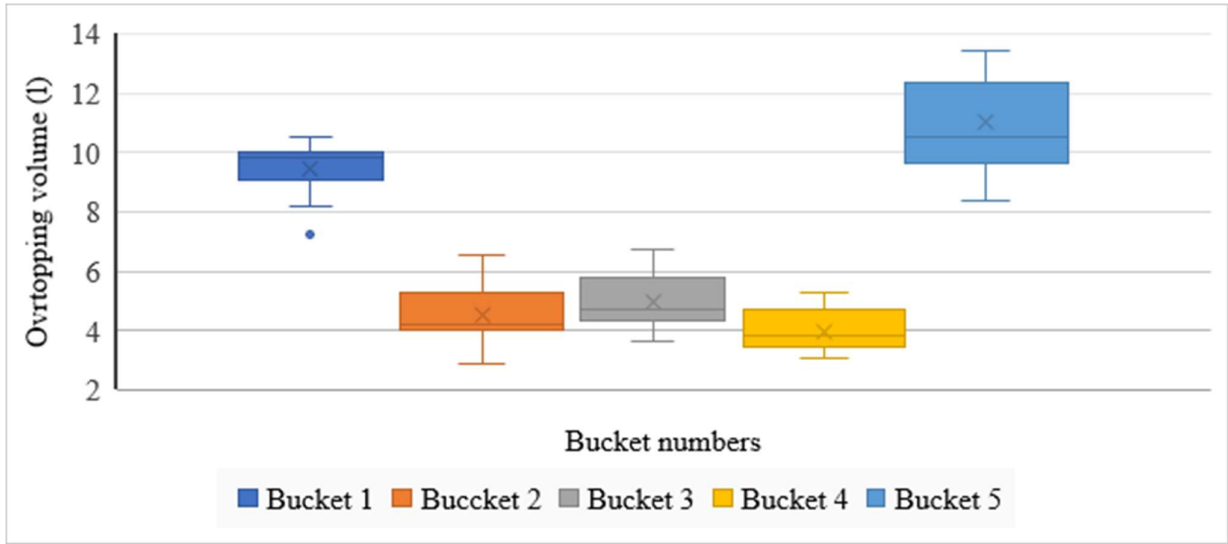


Figure 32: Box plot showing the outliers of the total overtopping volume measured in the five buckets.

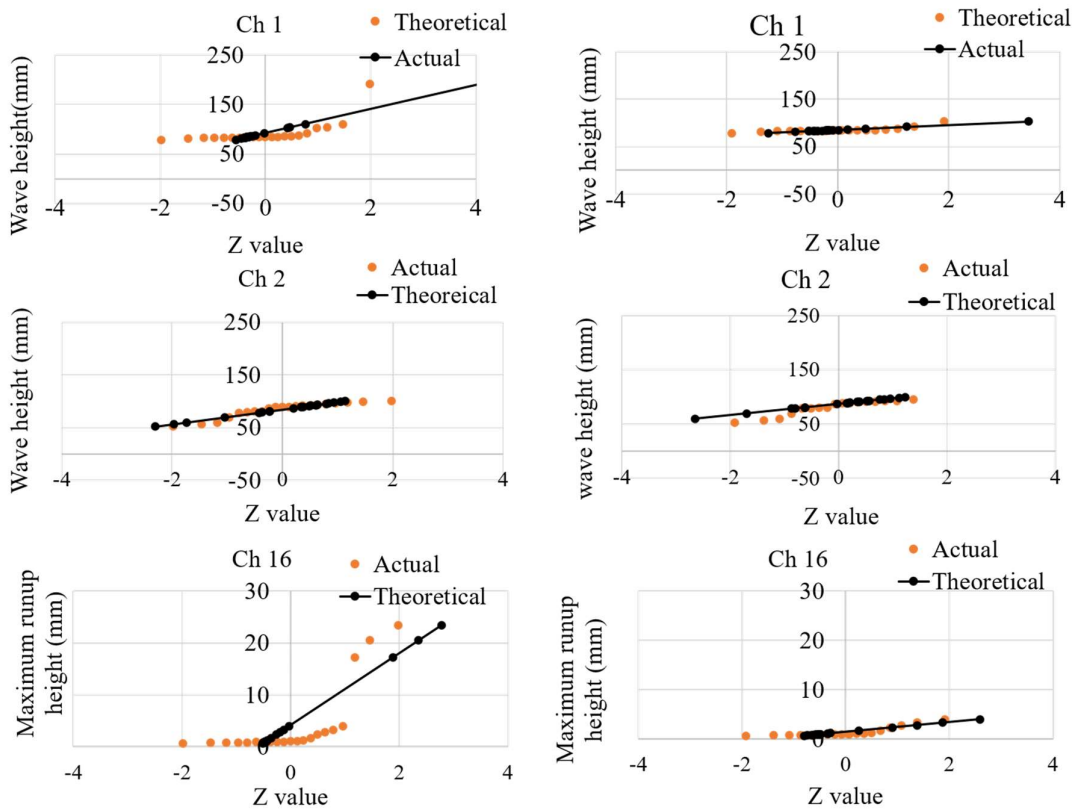


Figure 33: The plot between theoretical and actual sample data before and after removal of outliers for CH1, CH2 and CH16.

The experimental data recordings from CH4 has passed KS test but its graph doesn't resemble on normal distribution graph (see Figure 34).

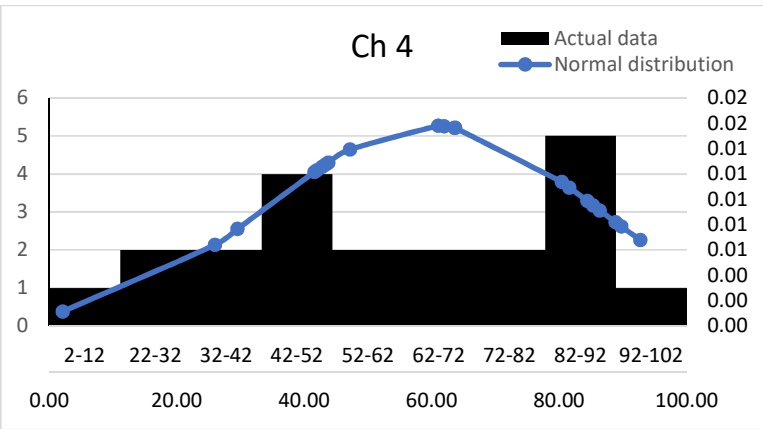


Figure 34: Normal distribution plot for CH4.

In addition, the wave height data recorded by this sensor doesn't have pattern in all 18 trials, noises have been noticed (Figure 35). Hence it is recommended that changing sensor 4 is mandatory for better results.

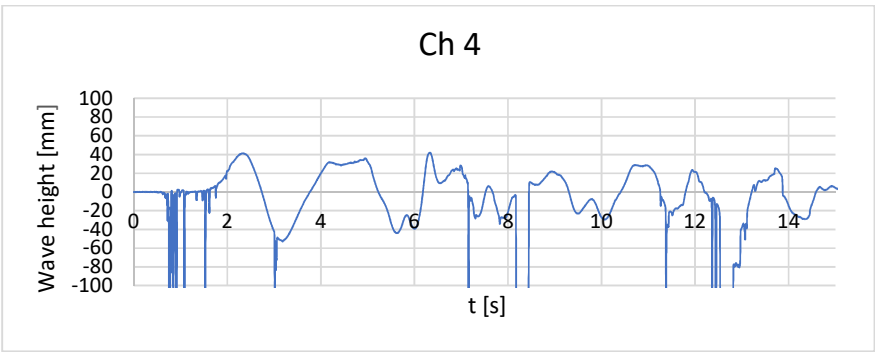


Figure 35: Plot for the wave height vs time for CH4.

5. Result and Discussion

This chapter focuses on the results from the experiments and then the discussion. In the first section, the wave propagation and the overtopping pattern is explained. Later the parameter, slide speed and overtopping volume is discussed in a separate section. Lastly the results from this study is compared to literature.

5.1. Wave propagation and overtopping pattern

The landslide impacts the water and transfers its kinetic energy to the water body. This displaces the water in the impact region primarily in the landslide motion direction, but also laterally around the landslide front. This displacement produces a radial wave crest that propagates away from the impact site (McFall and Fritz, 2016). This is also seen in the present study where after the slide blocks hit the still water, the water is pushed towards the opposite bank of the reservoir (opposite to the slide impact) and then towards the direction of the dam. Meanwhile, overtopping of the dam crest occurs if the runup height is greater than the freeboard provided for the dam.

In landslide generated waves, the wave propagation is mainly characterized by the maximum wave amplitude. For the discussion of the wave propagation process, the waves gauges are classified into three rows here as 1st row, 2nd row and 3rd row. The 1st row includes CH1, CH4 and CH7 and are located at the opposite side of the slide impact. The 2nd row of the wave gauges includes CH2, CH5 and CH8 and are located at the middle of the reservoir. The last row, 3rd row includes CH3, CH6 and CH9 and are located at the side of slide impact. The maximum wave amplitude for impact waves is recorded with CH1 which is located at the opposite side of the slide impact, in most of the experiments (see Figure 22 for the placement of the sensors). While the wave propagates along the reservoir, there is a wave decay so that the wave amplitude decreases through the reservoir. The wave height recorded for the nine wave gauges for the time duration of $t = 12\text{sec}$ is presented in Figure 36, Figure 37 and Figure

38. Referring to Figure 36, as the slide impacts the reservoir, the wave is generated and a wave height is first recorded at Ch 1. The maximum wave height $H = 79.2$ mm is recorded at $t = 1.81$ sec. Then the wave propagates through the reservoir and reaches CH4 at around $t = 1.5$ sec and the maximum wave crest $H = 39.3$ mm is recorded at $t = 2.47$. Consequently, the propagation continues and the wave reaches Ch7 and then overtops the dam crest. The maximum wave height $H = 29.3$ mm is recorded at $t = 3.2$ sec. The wave propagation pattern along the reservoir recorded by all the nine sensors classified as 1st row, 2nd row and 3rd row is illustrated in the Figure 36, Figure 37 and Figure 38 respectively.

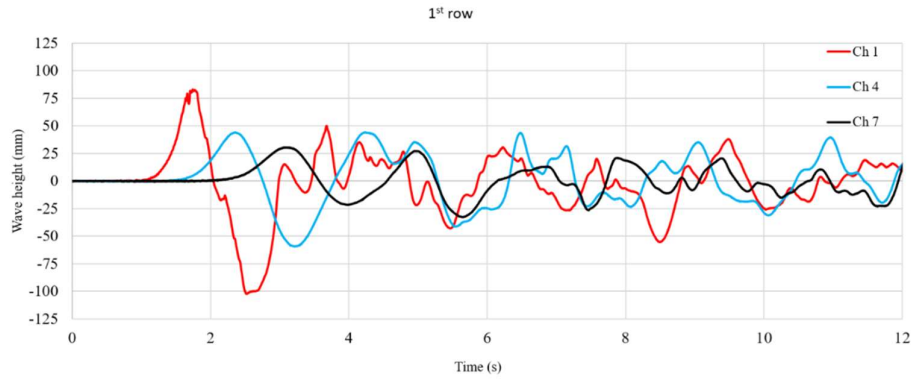


Figure 36: Plot for wave height recorded at the 1st row (CH1, CH4 and CH7) versus time (Test no. 288_2.25s_4.5_2H_150).

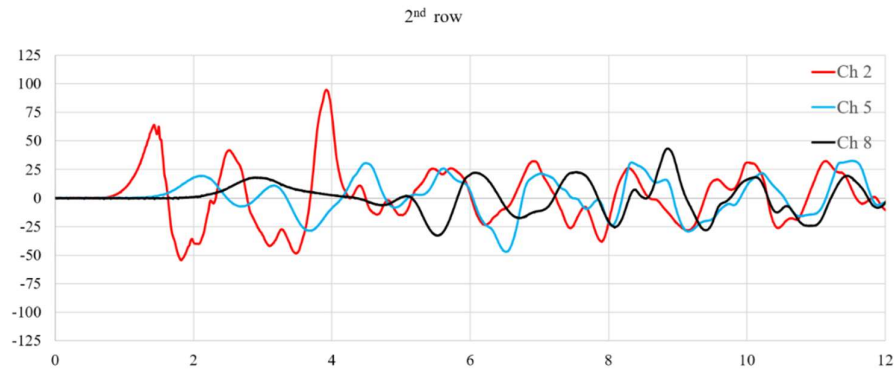


Figure 37: Plot for wave height recorded at the 2nd row (CH2, CH5 and CH8) versus time (Test no. 288_2.25s_4.5_2H_150).

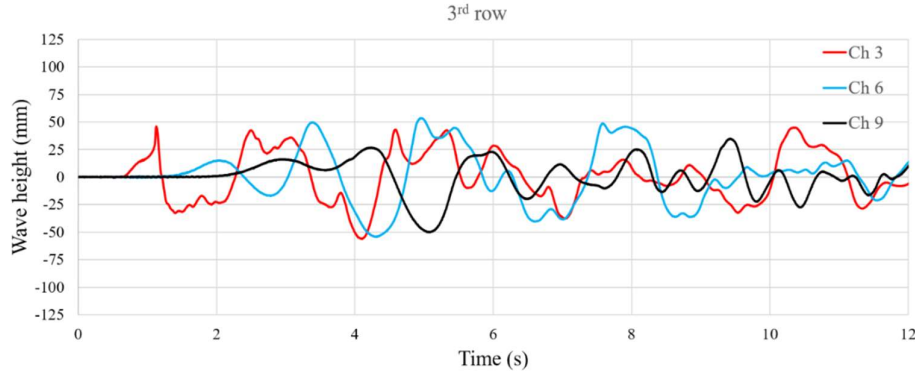


Figure 38: Plot for wave height recorded at the 3rd row (CH3, CH6 and CH9) versus time (Test no. 288_2.25s_4.5_2H_150).

Another interesting result from this study is the non uniform distribution of overtopping across the five dam crest sections. It is recalled that the dam crest is divided into five dam crest section to see the distribution of the overtopping along the dam crest length. This is illustrated with recording the overtopping depth distribution with CH11 to CH16, over the five dam crest sections. Figure 39 illustrates the overtopping depth with time with a specific test for a duration of $t = 14$ sec. Referring to Figure 39, the highest overtopping depth is recorded with the ultrasonic sensors located at the right and left edges of the dam crest (CH11 and CH16). This result can be attributed to the model setup used for this study, where reflections from the reservoir sides occur. In addition, the slide impinges perpendicular to reservoir's longitudinal axis. Furthermore, a histogram is made for the same test as that of Figure 39 . It clearly shows the overtopping distribution along the five dam crest sections (Figure 40). Accordingly, a maximum value among the five channels, overtopping depth of $d_o = 27.1$ mm and 31.1 mm is recorded for CH11 and CH16 respectively.

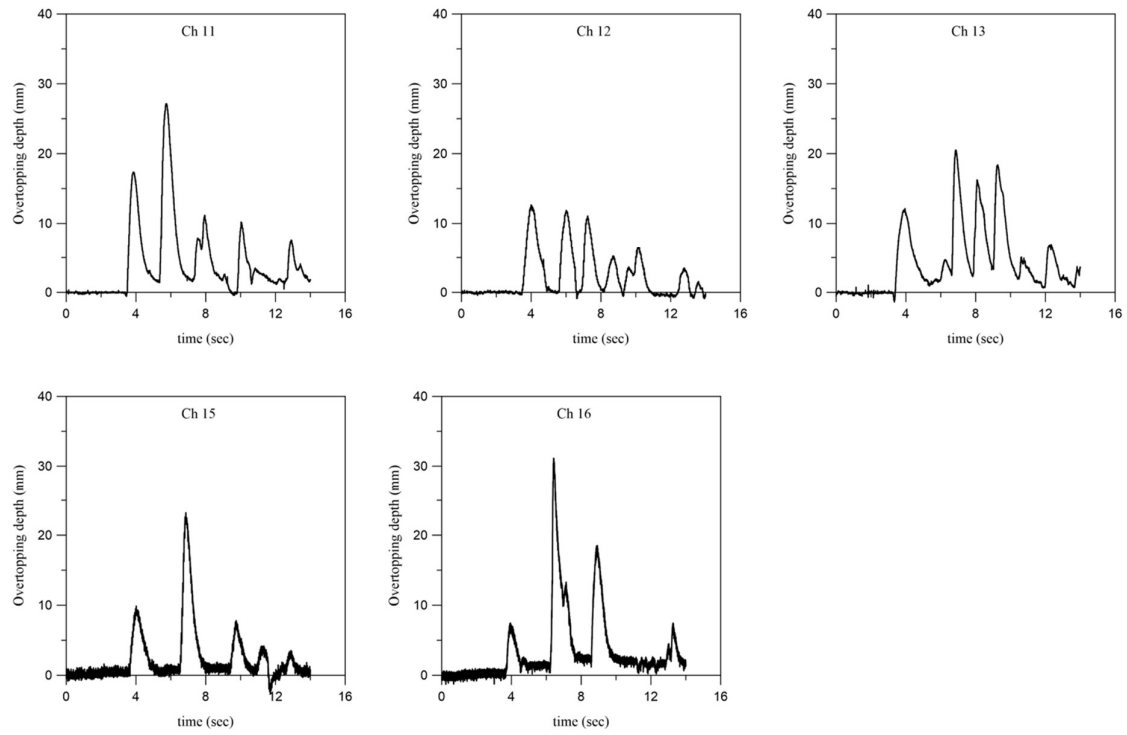


Figure 39: The overtopping pattern over the five dam crest sections (Test no. 188_2.25s_4.5m_2H_150).

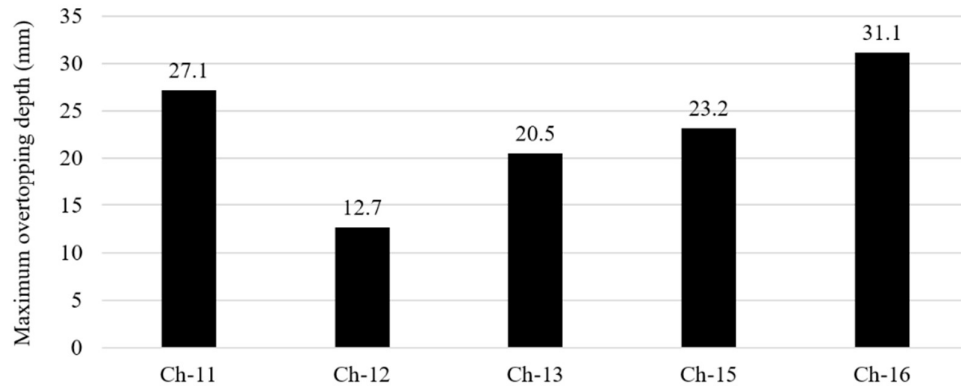


Figure 40: Histogram for overtopping depth distribution over the dam crest (Test no. 188_2.25s_4.5m_2H_150).

5.2. Slide impact velocity

The slide release height is one of the dominant parameters that will be used for defining the impact of landslide generating waves. It has a dominant impact on specifically on the slide impact velocity. Among the prediction methods in literature, Heller et al. (2009) presented a formula for determining slide impact velocity as;

$$V_s = \sqrt{2g\Delta z_{sc}(1 - \tan\delta \cot\alpha)} \quad 14$$

where V_s (m/s) = slide impact velocity;

g (m/s²) = gravitational acceleration;

α (°) = slide impact angle;

δ (°) = dynamic bed friction angle;

Δz_{sc} (m) = drop height of the center of gravity of the slide.

The drop height of the center of gravity of the slide is the vertical distance between the center of gravity of the slide mass in its original position and at the impact location. The dynamic bed friction angle represents the friction at the contact between the slide mass and the underlying slope, $\delta = 20^\circ$ is assumed.

In this model setup, the slide position versus time is recorded with position sensor (CH10). The maximum slide velocity is considered slide impact velocity in this study. Hence, in order to extract the maximum slide velocity certain steps are followed which will be discussed here in. First, the average speed, V_{avg} of the slide is calculated by;

$$V_{avg} = \frac{(s_2 - s_1)}{(t_2 - t_1)} \quad 15$$

The average speed has noises so that sixth order polynomial regression is applied to correct the noises, it is possible to filter the maximum slide speed (see Figure 41).

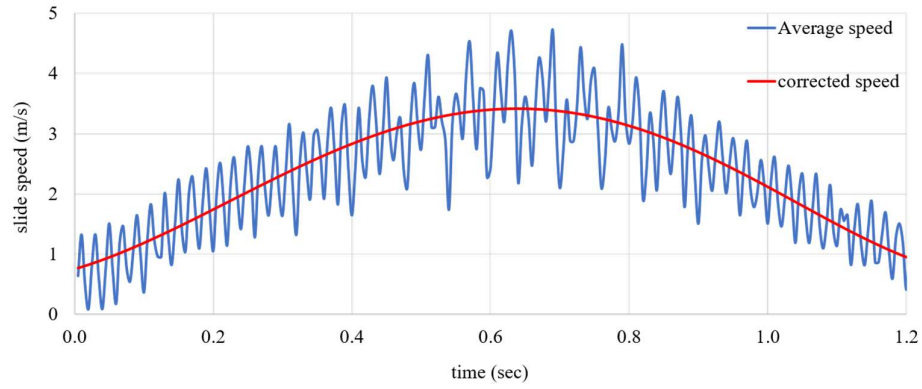


Figure 41: Average versus corrected speed for 203_2.25s_4.5_4_150.

The results from the experiments of this study confirms that, the initial position of the slide is the most dominant parameter to define the slide velocity. Increasing the travel distance of the slide to the water increases the value of the landslide speed significantly.

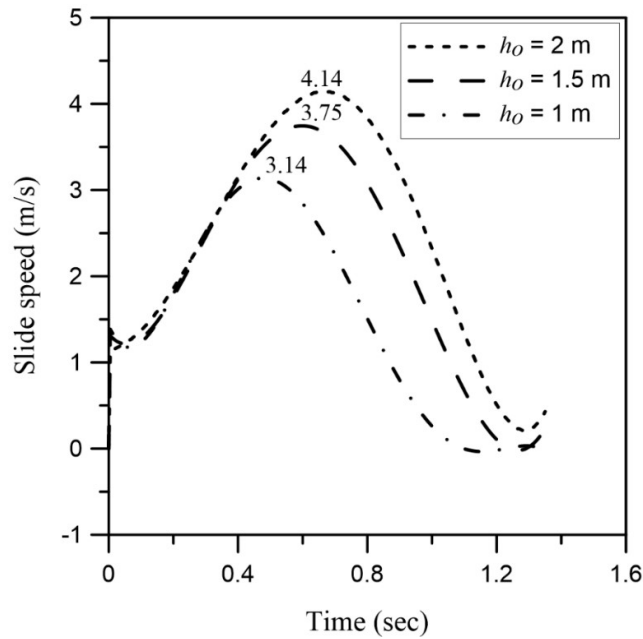


Figure 42: Slide velocity comparison considering similar setup for different release height.

The test numbers considered are: 185_2.25s_4.5_2H_200, 188_2.25s_4.5_2H_150 and 191_2.25s_4.5_2H_100.

In case of smaller travel distance, the impact velocity seems lower. This is illustrated with considering the same slide arrangement (2H blocks), $f = 4.5$ and $\beta = 2.25$ for different slide release height, $h_o = 2$ m, 1.5 m and 1 m (see Figure 42). The maximum slide speed recorded is $V_s = 4.14$ m/s, 3.75 m/s and 3.14 m/s for $h_o = 2$ m, 1.5 m and 1 m respectively (see Figure 42). It is clearly seen that the slide release height has an influence on slide speed. Accordingly, an increasing in the release height by a range of 0.5 m increases the slide speed in a factor of 84 % to 90 % for the same slide arrangement.

5.3. Overtopping volume

The distribution of the overtopping volume over the five buckets corresponding to each dam crest sections for a specific experimental setup is shown in Figure 43. The maximum overtopping volume is recorded at CH16 and then CH11 which are located at the right (where the slide impacts) and left section of the dam crest sections respectively (Refer to Figure 22 for the placement of the sensors along the dam crest). Similar result is observed in the overtopping depth distribution (section 5.2). This result can be attributed to the model setup used in this study, where reflection of waves from the reservoir sides occur. Most importantly the slide impinges into reservoir in perpendicular to reservoir's longitudinal axis.

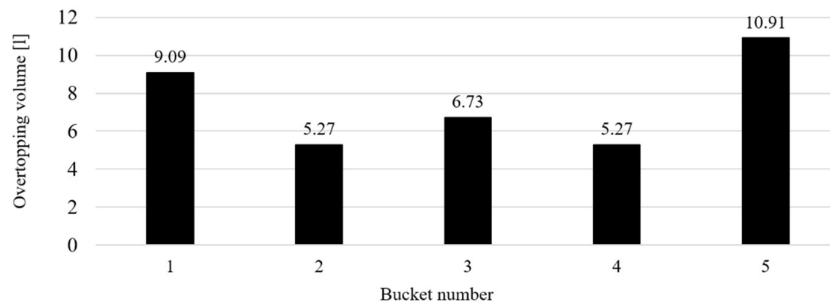


Figure 43: Overtopping volume distribution over the dam crest (Test no. 188_2.25s_4.5m_2H_150).

For a fixed upstream dam slope of 1: 1.5 and $h_o = 0.5$ m, the impact of slide volume on overtopping volume is investigated herein. Hence, the slide volume is found to have a significant effect on the overtopping volume (see Figure 44). This is illustrated by measuring the overtopping volume of water due to the different slide block arrangements having different slide volumes (2V, 4 and 6) falling into a reservoir. The 6-block arrangement, the highest block with volume of 0.225 m^3 , causes the highest overtopping volume. For this specific case (referring to Figure 44), the 2V and 6 block arrangements caused a maximum overtopping of 17.73 l and 66.91 l of water over the dam crest respectively.

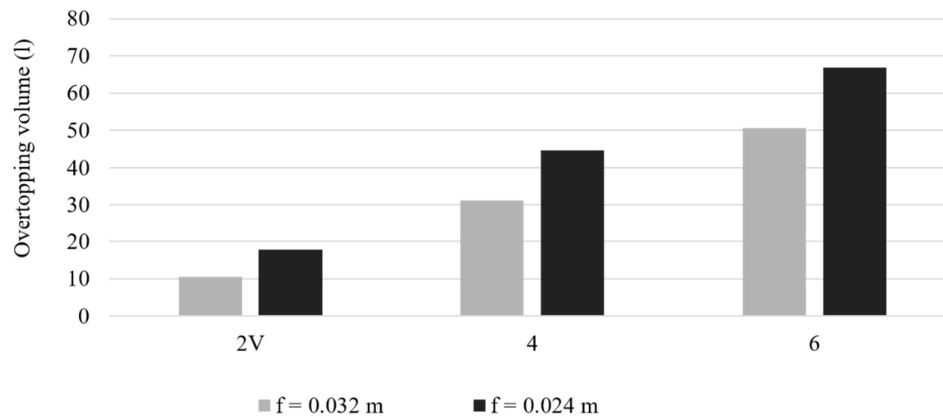


Figure 44: Overtopping volume for different slide arrangements ($\beta = 1: 1.5$ and $h_o = 0.5 \text{ m}$).

The freeboard, which is defined as the vertical distance between the top of the dam and reservoir full supply level, is another parameter which dominates the overtopping volume of water in landslide generated waves. The higher the freeboard provided for a dam, associated with the lower still water level in the reservoir, causes smaller overtopping volumes. For the same slide volume (for e.g. 2V block arrangement), the overtopping is recorded to be 10 l and 17.73 l for $f = 0.032$ and $f = 0.024 \text{ m}$ respectively (Figure 44). The same applies for the other block arrangements used in this study (2H, 4 and 6 blocks). The impact of both parameters, slide volume and freeboard is presented in Figure 45.

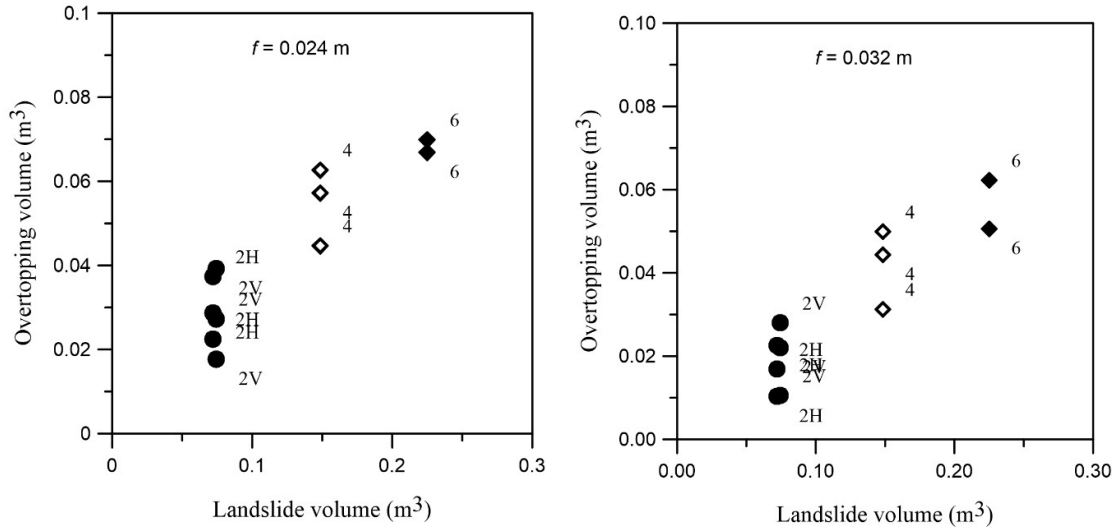


Figure 45: The figure shows the impact of slide volume and freeboard on overtopping volume.

It is clearly stated above that the slide volume influences the overtopping phenomenon in a way that larger overflows occur as this parameter increases. However, the slide impact velocity, which is directly influenced by the initial position of the slide also affects the overtopping. Indeed, the overtopping volume is lower in cases where the slide starting position on the ramp is closer to the still water level in the reservoir. This is illustrated in with considering 2V slide block arrangement with three different slide release height positions. For a specific configuration, an increase in slide release height from $h_o = 0.5$ m to 1 m increases the overtopping volume in a factor of 2.1 and 1.54 for $f = 0.032$ m and 0.024 m respectively. The same applies as h_o increases from 1m to 2m, the overtopping volume increases in a factor of 1.27 and 1.44 for $f = 0.032$ m and 0.024 m respectively.

The impact of upstream dam slope is considered in the analysis. The milder the dam slope becomes the more overtopping water flows over the dam crest. However, among all the parameters considered in this study, it can be concluded that the slide volume and freeboard are the two parameters which mostly define the overtopping volume. The slide impact velocity,

which is described based on the slide shape and release height, and the upstream dam slope has relatively minor effects.

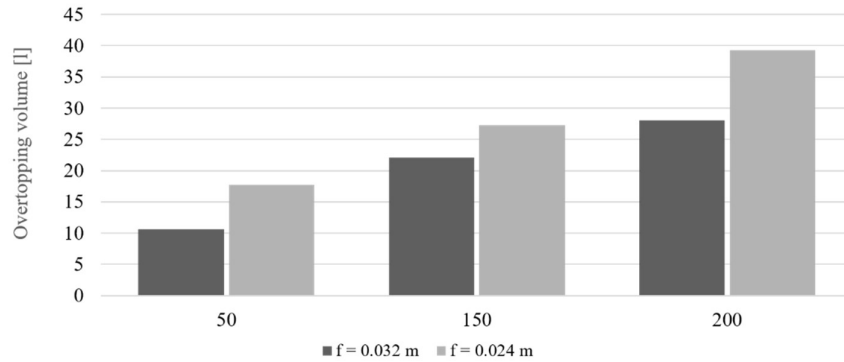


Figure 46: The relationship between slide volume and slide release height for 2V slide block arrangement.

5.4. Comparison to literature

In this section, the results from the present study is compared to the predictive equations from the literature. In the first sub-section (section 5.4.1), the impulse product parameter is applied for the results from this study: specifically, the wave amplitude and overtopping volume are discussed. In the second one (section 5.4.2), the predictive equations from Kobel et al. (2017) for the overtopping volume and overtopping depth are applied and compared to the measured value. The model setups used in Heller et al. (2007) and Kobel et al. (2017) are different from this study. However, in this section, whether the predictive equations from these literatures can be applied to the results from this study without modification is discussed.

5.4.1. Impulse product parameter

The impulse product parameter, P which is discussed in section 2.2 is applied here to define the maximum wave height and overtopping volume. The wave amplitude is recorded with the nine wave gauges installed on the top of the reservoir. The maximum among these values for each test is used for the analysis herein. For each test conducted in this study, P is first

calculated with Eq. (1), and then the maximum wave amplitude is predicted with Eq. (2). The comparison of the measured maximum wave amplitude a_M versus the predicted one with Eq. (2) is presented in Figure 47. The figure shows a good correlation between these values with coefficient of determination, $R^2 = 0.95$.

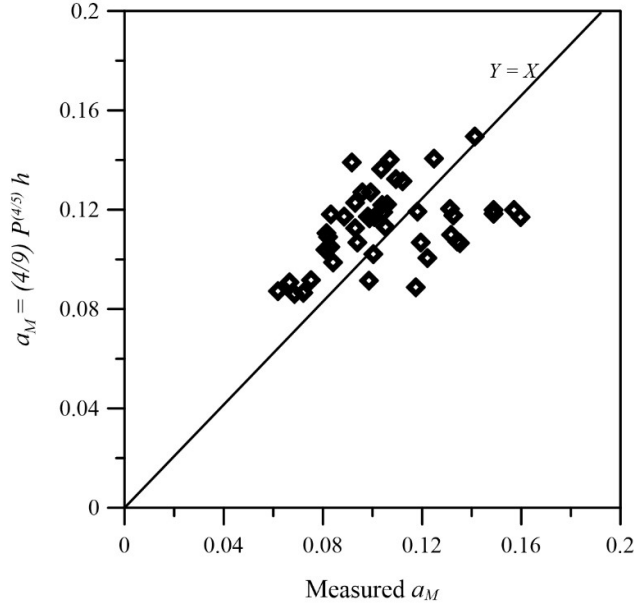


Figure 47: The comparison between the measured and predicted (with Eq. (2)) maximum amplitude.

Further, a comparison is made between the measured overtopping volume, W_W and impulse product parameter, P . Since P is dimensionless parameter, it would be appropriate to compare it to the dimensionless version of W_W . Hence, W_W is standardized with dividing by h^3 (still water depth cubic). The relationship between the two non dimensional parameters P and relative overtopping volume, W_W/h^3 is shown in Figure 48. However, there seems to be no trend between the two parameters. The reason for this can be attributed to impulse product parameter is defined for the wave generation and propagation phases of landslide generated waves. However, the overtopping volume is a parameter describing the third phase (i.e runup

and overtopping). In addition, P is defined for landslide generated waves in reservoirs in 2D laboratory scale model with granular slides, where the slide impinges in front of the dam.

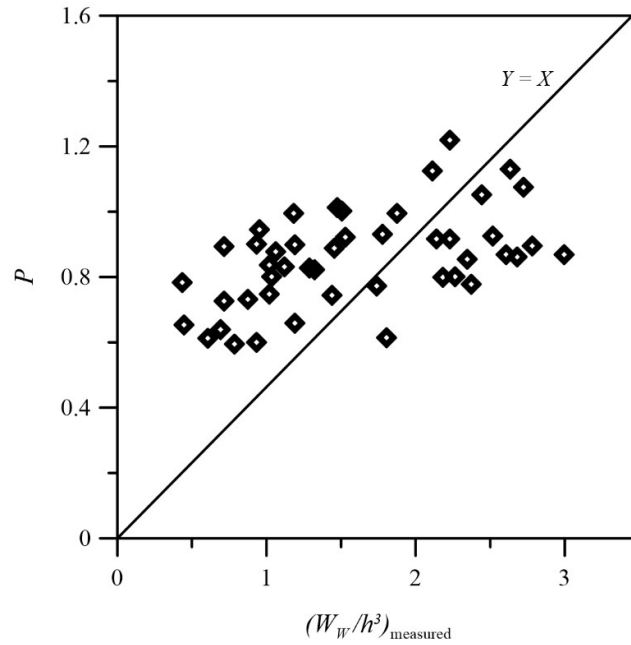


Figure 48: The figure shows the comparison of measured relative overtopping volume versus P .

5.4.2. Comparison to Kobel's et al., (2017) Equation

The methods for predicting the overtopping volume in the literature is described in the section 2.4 of this thesis. The measured relative overtopping volume from this study and the predicted one with Kobel's et al. (2017) Eq. (10) considering the effect of upstream dam slope is compared (Figure 49). The results from the predictive equation of Kobel's et al. (2017) over estimates the measured relative overtopping volume from the present study. The Figure 49 shows the minimum value of the overtopping volume predicted with Eq. (10) is 0.4.

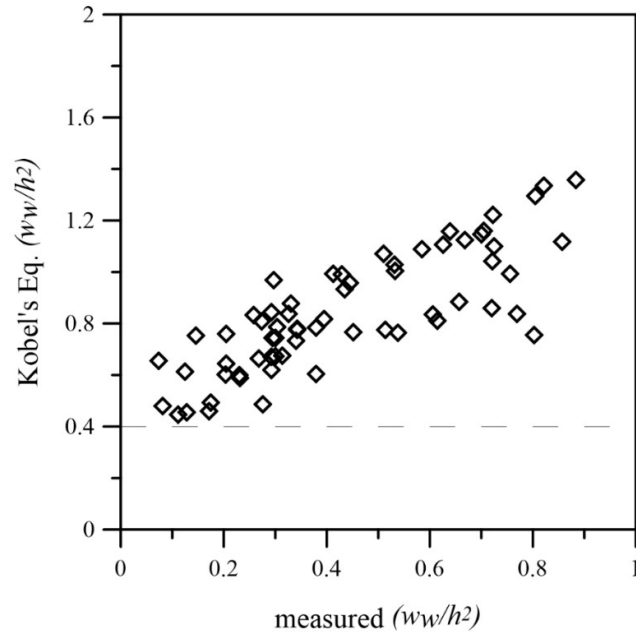


Figure 49: The comparison of measured and predicted relative overtopping volume.

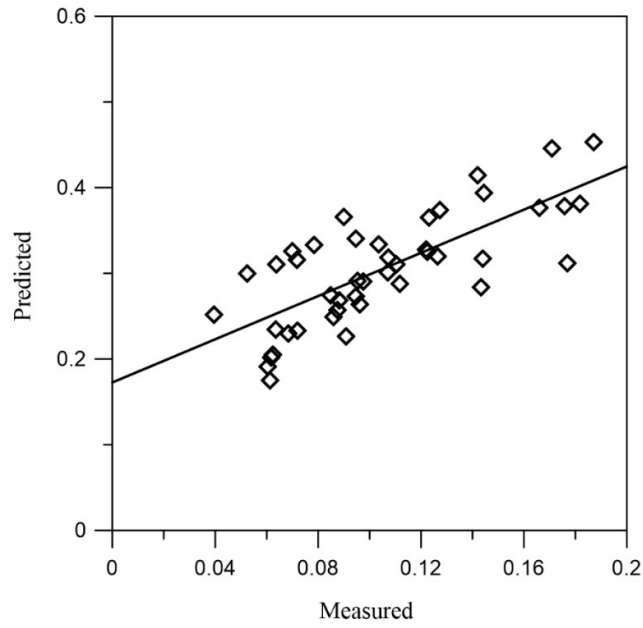


Figure 50: The comparison of measured and predicted (with Kobel's et al. (2017) Eq. (12)) relative overtopping depth.

Another parameter estimation which compared to literature is the overtopping depth. Kobel et al. (2017) presented two predictive equations with different R^2 values for the relative

overtopping depth (d_o/w) due slide generated waves. The measured overtopping depth is first normalized with the dam height(d_o/H), so that a comparison is made to the results from Kobel's et al. (2017) Eq. (11) (Figure 50). The results show that the predicted value from the Kobel's et al. (2017) over estimates the measured relative overtopping depth in a range of 1.8 to 6.4.

6. Conclusion and Recommendation

6.1. Conclusion

This thesis investigates the overtopping of a dam by landslide generated waves through physical model tests. The model produces a typical dam reservoir with unstable hillsides i.e. the slides falling into the reservoir's longitudinal axis. A comparison is made between the measured data from the present study and to literature. The main conclusions from this study are:

- Among the different slide arrangements, 6 block arrangement with volume of 0.225 m^3 (model scale) caused a maximum overtopping volume.
- The slide volume is the most dominant one among the parameters followed by freeboard and slide release height that influences the overtopping volume. The relationship between slide volume and overtopping volume seems linear. For a 2V block arrangement, a decrease in freeboard from $f = 0.032$ and $f = 0.024$ m increases the overtopping volume from 10 l to 17.73 l . For a specific configuration, an increase in slide release height in a range of 0.5 m and 1 m increases the overtopping volume in a range 1.27 to 2.1 for $f = 0.032 \text{ m}$ and 0.024 m respectively. The landslide release height can be expressed in terms of slide impact velocity. However, the impact of upstream dam slope angle seems minor relative to other parameters.
- Landslide release height, the initial position of the slide relative to still water level, is the most dominant parameter that influences the landslide speed. This is illustrated through for a fixed setup, an increase in the slide release height by a range of 0.5 m increases the slide speed in a factor of 84% to 90% .
- There is a good correlation between the measured maximum wave amplitude and the predicted one based on impulse product parameter with $R^2 = 0.95$.

- There is no trend between the impulse product parameter, P and the overtopping volume.
- The equations from Kobel's et al. (2017) over estimates the measured overtopping volume and overtopping depth values.
- The overtopping volume and overtopping depth are not uniformly distributed along the dam crest sections for this specific model setup. The maximum value is observed at the right and left edges of the dam crest section.

In addition to this, qualitative analysis of the wave overtopping process was investigated and found that the overtopping process is not uniform all over the dam crest sections. Maximum overtopping has been recorded at the right and left edge (CH11 and CH16) of the dam crest sections. The same applies for the overtopping depth. This can be attributed to the 3D model test considered in this study, where the landslide impinges perpendicular to the reservoir's longitudinal axis.

6.2. Recommendation

The equations describing the impulse wave generated waves are mostly derived based on physical model test on rectangular flume. In this scenario, the slide impacts directly to the reservoir and then the dam. Hence, it would be interesting to explore the case of impulse wave generated waves based on wave basin, where the landslide impacts perpendicular to reservoir's longitudinal axis as in the case considered in this study. This is common in most real cases.

The following recommendations are given:

- In design of new dams in alpine regions, the freeboard design should consider the impact of slide generated waves.

- In national guideline for design of dams which is applicable in Ethiopia, the design steps and calculation procedure for the design of freeboard against landslide generated waves should be clearly stated.
- All dams constructed in alpine regions or slide susceptible areas should consider analyzing the impact of slides if they fall. This helps in providing preventive measures and emergency preparedness.
- A slide susceptibility map for all reservoir dams constructed in Ethiopian should be prepared.
- The topic landslide generated waves in reservoirs and impacts in dams is barely investigated and studied in Ethiopia. Hence, better understanding should appear through research works for example in master's level.

References

- Abebe, B., Dramis, F., Fubelli, G., Umer, M., and Asrat, A. (2010). Landslides in the Ethiopian highlands and the Rift margins. *Journal of African Earth Sciences* 56, 131–138.
- Aman, A., Mammo, T., and Wieland, M. (2019). Seismic safety evaluation of Tekeze arch dam. pp. 2635–2648.
- Ataie-Ashtiani, B., and Malek-Mohammadi, S. (2007). Near field amplitude of sub-aerial landslide generated waves in dam reservoirs. *Dam Eng* 17, 197–222.
- Ayalew, L. (1999). The effect of seasonal rainfall on landslides in the highlands of Ethiopia. *Bull Eng Geol Env* 58, 9–19.
- Barla, G., and Paronuzzi, P. (2013). The 1963 Vajont Landslide: 50th Anniversary. *Rock Mech Rock Eng* 46, 1267–1270.
- Bosa, S., and Petti, M. (2011). Shallow water numerical model of the wave generated by the Vajont landslide. *Environmental Modelling & Software* 26, 406–418.
- Broothaerts, N., Kissi, E., Poesen, J., Van Rompaey, A., Getahun, K., Van Ranst, E., and Diels, J. (2012). Spatial patterns, causes and consequences of landslides in the Gilgel Gibe catchment, SW Ethiopia. *CATENA* 97, 127–136.
- Cruden, D.M., and Varnes, D.J. (1996). *Landslide Types and Processes* (National Academy of Sciences).
- Di Risio, M., and Sammarco, P. (2008). Analytical Modeling of Landslide-Generated Waves. *Journal of Waterway, Port, Coastal, and Ocean Engineering* 134, 53–60.

- Dickson, P., and Loar, T.N. (2011). Abutment Stability Analyses and Design Solutions for Tekeze Dam. *ARMA 11*.
- Fritz, H. (2002). PIV applied to landslide generated impulse waves. pp. 305–320.
- Fritz, H.M., Hager, W.H., and Minor, H.-E. (2003). Landslide generated impulse waves. *Experiments in Fluids* 35, 505–519.
- Gallati, M., Braschi, G., and Falappi, S. (2005). SPH simulations of the waves produced by a falling mass into a reservoir. *Nuovo Cimento C Geophysics Space Physics C* 28, 129.
- Genevois, R., and Ghirotti, M. (2005). The 1963 Vaiont Landslide. *Giornale Di Geologia Applicata* 1, 41–52.
- Hailemariam Gugsu, T., and Schneider, J.F. (2010). Rock Mass Classification of Karstic Terrain in the Reservoir Slopes of Tekeze Hydropower Project. p. 831.
- Hall and Watts (1953). Laboratory investigation of the vertical rise of solitary waves on impermeable slopes.
- Heller, V. (2007). Landslide generated impulse waves - Prediction of near field characteristics. *ETH/VAW* 177.
- Heller, V., and Hager, W.H. (2010). Impulse Product Parameter in Landslide Generated Impulse Waves. *Journal of Waterway, Port, Coastal, and Ocean Engineering* 136, 145–155.
- Heller, V., and Hager, W.H. (2011). Wave types of landslide generated impulse waves. *Ocean Engineering* 38, 630–640.
- Heller, V., Hager, W.H., and Minor, H.-E. (2009). *Landslide Generated Impulse Waves in Reservoirs: Basics and Computation* (ETH Zürich).

Huber, A. (1980). Schwallwellen in Seen als Folge von Felsstürzen.

Huber, A. (1982). Impulse waves in Swiss lakes as a result of rock avalanches and bank slides, Experimental results for the prediction of the characteristic numbers of these waves. Trans. 14th Int. Congress Large Dams 3, 455–476.

Huber, A., and Hager, W.H. (1997). Forecasting impulse waves in reservoirs. Proc. 19th Congrès Des Grands Barrages 993–1005.

Kamphuis, J.W., and Bowering, R.J. (1970). Impulse waves generated by landslides. Coastal Engineering Proceedings 1.

Kobel, J., Evers, F.M., and Hager, W.H. (2017). Impulse Wave Overtopping at Rigid Dam Structures. Journal of Hydraulic Engineering 143, 04017002.

Le Mehaute, B. (1976). Similitude in Coastal Engineering. Journal of the Waterways, Harbors and Coastal Engineering Division 102, 317–335.

Lysne, D., Glover, B., Stole, H., and Einar, T. NTNU Book Series- Hydropower Development (NTNU).

Manenti S., Pierobon E., Gallati M., Sibilla S., D’Alpaos L., Macchi E., and Todeschini S. (2016). Vajont Disaster: Smoothed Particle Hydrodynamics Modeling of the Postevent 2D Experiments. Journal of Hydraulic Engineering 142, 05015007.

McFall, B.C., and Fritz, H.M. (2016). Physical modelling of tsunamis generated by three-dimensional deformable granular landslides on planar and conical island slopes. Proc. Math. Phys. Eng. Sci. 472, 20160052.

- Miller, D.J. (1960). Giant Waves in Lituya Bay Alaska. Geological Survey Professional Paper 48.
- Mueller, L. (1968). New considerations of the Vaiont slide. Rock Mechanics & Engineering Geology.
- Mueller D, and Schurter M (1993). Impulse waves generated by an artificially induced rockfall in a Swiss lake. Proceedings of Congress of International Association for Hydraulic Research 25th, 209–216.
- Müller, D.R. (1995). Auflaufen und Überschwappen von Impulswellen an Talsperren (ETH Zürich).
- Noda, E. (1970). Water waves generated by landslides. Journal of the Waterways, Harbors and Coastal Engineering Division 96, 835–855.
- Novák, P. (1981). Models in Hydraulic Engineering: Physical Principles and Design Applications (Boston: Pitman).
- Panizzo, A., de Girolamo, P., and Petaccia, A. (2005). Forecasting impulse waves generated by subaerial landslides. Journal of Geophysical Research (Oceans) 110, C12025.
- Sælevik, G., Jensen, A., and Pedersen, G. (2009). Experimental investigation of impact generated tsunami; related to a potential rock slide, Western Norway. Coastal Engineering 56, 897–906.
- Slingerland, R.L., and Voight, B. (1979). Occurrences, Properties, and Predictive Models of Landslide-Generated Water Waves. In Developments in Geotechnical Engineering, B. Voight, ed. (Elsevier), pp. 317–394.

- Steven, N.W., and Simon, D. (2011). The 1963 Landslide and Flood at Vaiont Reservoir Italy. A tsunami ball simulation. *Italian Journal of Geosciences* *130*.
- Synolakis, C.E. (1987). The runup of solitary waves. *Journal of Fluid Mechanics* *185*, 523–545.
- Walder, J.S., Watts, P., Sorensen, O.E., and Janssen, K. (2003). Tsunamis generated by subaerial mass flows. *Journal of Geophysical Research: Solid Earth* *108*.
- Wiegel, R.L., Noda, E.K., Kuba, E.M., Gee, D.M., and Tornberg, G.F. (1970). Water Waves Generated by Landslides in Reservoirs. *Journal of the Waterways, Harbors and Coastal Engineering Division* *96*, 307–333.
- Yalin, M.S. (1971). Principles of the Theory of Similarity. In *Theory of Hydraulic Models*, M.S. Yalin, ed. (London: Macmillan Education UK), pp. 35–50.
- Zhao, L., Mao, J., Bai, X., Liu, X., Li, T., and Williams, J.J.R. (2016). Finite element simulation of impulse wave generated by landslides using a three-phase model and the conservative level set method. *Landslides* *13*, 85–96.
- Zweifel, A., Willi H., H., and Hans-Erwin, M. (2006). Plane Impulse Waves in Reservoirs. *Journal of Waterway, Port, Coastal, and Ocean Engineering* *132*.
- Zweifel, A., Zuccalà, D., and Gatti, D. (2007). Comparison between Computed and Experimentally Generated Impulse Waves. *Journal of Hydraulic Engineering* *133*, 208–216.