Hazard assessment of debris flows initiated by breaching of small earth dams

by

Seyed Amirali Mehdizadeh

M.A.Sc., University of Tehran, 2010

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF
THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

in

The Faculty of Graduate Studies

(Civil Engineering)

THE UNIVERSITY OF BRITISH COLUMBIA

(Okanagan)

March 2013

© Seyed Amirali Mehdizadeh, 2013
Abstract

There are many small earth dams perched high above the floor of the Okanagan Valley in British Columbia. These dams pose a potential risk for destructive debris flow generation if they become breached. A relatively small outburst can trigger a much larger volume debris flow downstream of the dam. The failure of the Testalinden dam in the southern Okanagan region in June 2010 clearly demonstrated the destructive power of a debris flow triggered by the water released by a breach through a poorly maintained dam. Homes were destroyed and property was damaged. This thesis presents a methodology for preliminary assessment of potential debris flow initiation hazards caused by the breaching of small earth dams using digital elevation models, available maps, and limited monitoring records of dams. Research can assist dam safety officers in better ranking the consequences of dam failure. Empirical equations are used to predict the peak outflow if a breach occurs in a small earth dam. The creek gradient and the estimated height of water or outflow per unit width in the creek channel resulting from the outflow are used in debris flow initiation criteria to delineate possible locations along a creek where a debris flow may initiate. If debris flow initiation were possible, this would trigger the need for more detailed assessment of dam failure consequences and will likely result in a higher dam failure consequence classification compared to consideration of flooding only.
# Table of Contents

Abstract .......................................................................................................................... ii

Table of Contents ........................................................................................................... iii

List of Figures .................................................................................................................. v

List of Tables .................................................................................................................... viii

Acknowledgements ......................................................................................................... ix

Dedication ......................................................................................................................... x

Chapter 1 Introduction ..................................................................................................... 1
  1.1 Objectives ............................................................................................................... 3

Chapter 2 Earth Dams ...................................................................................................... 4
  2.1 Earth dams .............................................................................................................. 4
    2.1.1 Definition of small earth dams ....................................................................... 4
    2.1.2 Modes of earth dam failure .......................................................................... 5
    2.1.3 Small dam issues ......................................................................................... 6
  2.2 Dams in BC ........................................................................................................... 8
    2.2.1 Study area and case study ............................................................................ 9
  2.3 Dam safety evaluation .......................................................................................... 11
    2.3.1 Standards-based approach ........................................................................... 11
    2.3.2 Risk-based approach ................................................................................... 11
  2.4 Dam failure consequences ................................................................................... 12
    2.4.1 Life safety ................................................................................................... 12
    2.4.2 Economic impacts ....................................................................................... 13
    2.4.3 Environmental and cultural losses ............................................................... 13
  2.5 Dam breach peak outflow and hydrograph ......................................................... 13
    2.5.1 Empirical equations for peak outflow, $Q_p$ ............................................... 14
    2.5.2 Dam breach hydrograph ............................................................................. 15
    2.5.3 Empirical equations for time of failure, $t_f$ .............................................. 17

Chapter 3 Debris Flow Initiation .................................................................................... 19
  3.1 Debris flows and mud flows ................................................................................. 19
    3.1.1 Development of debris flow due to erosion of a gully bed ....................... 20
    3.1.2 Landslide-induced debris flow ................................................................... 20
    3.1.3 Debris flow induced by the collapse of a dam ........................................... 20
  3.2 Debris flow initiation thresholds .......................................................................... 21
    3.2.1 Takahashi (2007) ...................................................................................... 21
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.2.2</td>
<td>Berti and Simoni (2003)</td>
<td>23</td>
</tr>
<tr>
<td>3.2.3</td>
<td>Yu (2011)</td>
<td>27</td>
</tr>
<tr>
<td>3.3</td>
<td>Sources of data for predicting debris flow initiation</td>
<td>27</td>
</tr>
<tr>
<td>3.3.1</td>
<td>Dam breach hydrograph</td>
<td>27</td>
</tr>
<tr>
<td>3.3.2</td>
<td>Creek gradient and cross-section</td>
<td>28</td>
</tr>
<tr>
<td>3.3.3</td>
<td>Surficial geology and creek bed sediments</td>
<td>29</td>
</tr>
<tr>
<td>3.4</td>
<td>Tools for evaluation of debris flow hazard</td>
<td>29</td>
</tr>
<tr>
<td>3.4.1</td>
<td>Map analysis</td>
<td>30</td>
</tr>
<tr>
<td>3.4.2</td>
<td>Aerial reconnaissance</td>
<td>30</td>
</tr>
<tr>
<td>3.4.3</td>
<td>Field reconnaissance</td>
<td>30</td>
</tr>
<tr>
<td>3.4.4</td>
<td>Computerized debris flow analysis</td>
<td>30</td>
</tr>
<tr>
<td>3.4.5</td>
<td>Methodology for assessing debris flow initiation</td>
<td>30</td>
</tr>
<tr>
<td>Chapter 4</td>
<td>Testalinden Dam Case Study</td>
<td>33</td>
</tr>
<tr>
<td>4.1</td>
<td>Testalinden Lake and Dam</td>
<td>35</td>
</tr>
<tr>
<td>4.2</td>
<td>Testalinden Creek</td>
<td>36</td>
</tr>
<tr>
<td>4.2.1</td>
<td>Surficial geology of Testalinden Creek</td>
<td>41</td>
</tr>
<tr>
<td>4.3</td>
<td>Climate near Testalinden Dam</td>
<td>42</td>
</tr>
<tr>
<td>4.3.1</td>
<td>Oliver weather station</td>
<td>42</td>
</tr>
<tr>
<td>4.3.2</td>
<td>Mount Kobau Weather station</td>
<td>44</td>
</tr>
<tr>
<td>4.4</td>
<td>Testalinden Dam breach</td>
<td>47</td>
</tr>
<tr>
<td>4.4.1</td>
<td>Testalinden breach peak outflow and hydrograph</td>
<td>49</td>
</tr>
<tr>
<td>4.5</td>
<td>Analysis of debris flow initiation</td>
<td>50</td>
</tr>
<tr>
<td>4.5.1</td>
<td>Discussion of Testalinden debris flow initiation results</td>
<td>57</td>
</tr>
<tr>
<td>4.6</td>
<td>Sensitivity analysis</td>
<td>59</td>
</tr>
<tr>
<td>Chapter 5</td>
<td>Glanzier Case Study</td>
<td>61</td>
</tr>
<tr>
<td>5.1</td>
<td>Glanzier Creek watershed and Glanzier Dam</td>
<td>61</td>
</tr>
<tr>
<td>5.2</td>
<td>Creek gradient and cross-section</td>
<td>63</td>
</tr>
<tr>
<td>5.1</td>
<td>Surficial geology and creek bed sediments</td>
<td>64</td>
</tr>
<tr>
<td>5.2</td>
<td>Glanzier Dam breach hydrograph</td>
<td>66</td>
</tr>
<tr>
<td>5.3</td>
<td>Analysis of debris flow initiation</td>
<td>67</td>
</tr>
<tr>
<td>Chapter 6</td>
<td>Conclusions</td>
<td>71</td>
</tr>
<tr>
<td>References</td>
<td></td>
<td>73</td>
</tr>
</tbody>
</table>
List of Figures

Figure 2.1 Earth dam types ........................................................................................................ 4
Figure 2.2 Overtopping and piping failure modes for an earth dam (Wu 2011) ...................... 6
Figure 2.3 Dam types in BC ..................................................................................................... 8
Figure 2.4 Earth dams in the Okanagan Region ....................................................................... 10
Figure 2.5 Dam risk assessment procedure (modified from Mufute et al. 2008) .................... 12
Figure 2.6 Simple triangular hydrograph ............................................................................. 17
Figure 3.1 Shear stress distribution ........................................................................................ 22
Figure 3.2 Schematic of creek model (modified from Berti and Simoni 2003) ....................... 24
Figure 3.3 Soil phase diagram for creek sediment and resulting debris flow ....................... 26
Figure 3.4 Digital elevation model ........................................................................................ 28
Figure 3.5 Methodology to assess debris flow initiation caused by dam failure .................... 32
Figure 4.1 Topographic map of Testalinden Creek (modified from Atlas of Canada 2012) .... 33
Figure 4.2 Debris flow deposit on Testalinden Creek fan (photo credit: Dwayne Tannant) .... 34
Figure 4.3 Testalinden Lake (photo credit: Dwayne Tannant) ................................................ 36
Figure 4.4 Testalinden creek gradient .................................................................................... 38
Figure 4.5 Testalinden Creek and its longitudinal profile ....................................................... 39
Figure 4.6 Assumed schematic cross-section of the sediments
    in the Testalinden Creek before the debris flow ................................................................. 41
Figure 4.7 Surficial geology map of Testalinden Creek .......................................................... 42
Figure 4.8 Daily total precipitation for June 2010 at Oliver Airport weather station .......................... 43

Figure 4.9 Maximum and minimum daily temperature
in 2010 at Oliver Airport weather station ................................................................. 44

Figure 4.10 Monthly total precipitation at the Oliver station in 2010 ........................................... 44

Figure 4.11 Mean, maximum, and minimum monthly temperature
at the Mount Kobau weather station (1970 to 1980) ................................................. 45

Figure 4.12 Average monthly precipitation and snow fall
at the Mount Kobau weather station (1970 to 1980) .................................................. 46

Figure 4.13 Mean monthly snow pack at the Mount Kobau snow pack station
located at an elevation of 1817 m (1966 to 2012) ......................................................... 46

Figure 4.14 Breach through Testalinden Dam (photo credit: Dwayne Tannant) ......................... 47

Figure 4.15 Surveyed breach cross section (Tannant 2010) ..................................................... 48

Figure 4.16 Testalinden breach photo, look upstream (photo credit: Dwayne Tannant) ............... 49

Figure 4.17 Simplified cross-section of Testalinden Dam breach ............................................ 49

Figure 4.18 Simplified predicted hydrograph for the Testalinden Dam breach ......................... 50

Figure 4.19 Debris flow initiation predicted to occur
in the upper reach of Testalinden Creek ......................................................................... 55

Figure 4.20 Boulders carried by the debris flow (Hayduk 2010) ............................................... 57

Figure 4.21 Probable debris flow initiation location .................................................................... 58

Figure 4.22 Evidence of the passage of the debris flow .............................................................. 59

Figure 4.23 Slope distribution for Reach 2 ................................................................................ 60

Figure 5.1 Glanzier Creek (modified from Atlas of Canada 2012) ............................................. 62
Figure 5.2 Glanzier Dam (photo credit: Ministry of Forests, Lands and Natural Resource Operations, BC) ........................................................................................................ 63

Figure 5.3 Glanzier Creek and its longitudinal profile ........................................................................................................ 65

Figure 5.4 Surficial geology map of the Glanzier Creek region .......................................................... 66

Figure 5.5 Estimated hydrograph for a breach of Glanzier Dam .......................................................... 67

Figure 5.6 Boulders in the Glanzier Creek channel downstream of the dam (photo credit: Ministry of Forests, Lands and Natural Resource Operations, BC) ................. 70
List of Tables

Table 1.1 Dam classification (CDA 2007) .......................................................... 2

Table 2.1 Dam failures in the Okanagan valley (Tannant and Skermer 2011) ............... 8

Table 2.2 Selected empirical equations of peak outflow through a dam breach ................ 15

Table 2.3 Selected empirical equations for estimating time of dam failure ....................... 18

Table 4.1 Testalinden Creek debris flow initiation based on Equation 5 and 6 .................. 52

Table 4.2 Testalinden Creek debris flow initiation based on Equation 7 ......................... 54

Table 4.3 Particle diameter mobilized by channel discharge ....................................... 56

Table 5.1 Glanzier Creek reach properties and predicted factor of safety for debris flow initiation .......................................................... 69
Acknowledgments

Working on M.A.Sc. has been wonderful and often overwhelming experience. It is hard to say whether is grappling with the topic itself that has been real learning experience or grappling with how to write proposals, ask for data, and stay focused.

In any case, I am indebted to many people for assisting me during my thesis.

First, I am deeply grateful to my supervisor Dr. Dwayne D. Tannant. Working with you has been a real pleasure for me, and I have gained lots of experience. You have always been patient and encouraging of new ideas and difficulties no matter how busy your schedule is. Discussion with you led to key insights.

In addition, I would like to acknowledge engineers and offices that share data and give comments to us: Mike Noseworthy, Nigel Skermer, Shelley Higman, Jason Pither, and Maija Finvers, data collection would not have been possible without these individuals.
To my Mother and Father
Chapter 1 Introduction

There are approximately 10,000 small earth dams in Canada, many of which are older than their design life (Grapple and Mitchelmore 2009). As these dams continue to age and as development continues in areas downstream, dam failures will become a more significant public safety issue. Worldwide, much attention is given to prevent the failure of medium to large size dams with little attention being paid to small dams (Pisaniello et al. 2006). Various agencies are working to improve criteria used to assess risks associated with dam failures, and Canadian provinces have implemented new legislation or are considering changes to existing legislation to improve dam management. Little attention is normally given to the risk of failure of small dams because dam failures are normally viewed in the context of the risk that is posed to life and property downstream of the dam caused by the flood triggered from a dam collapse. Floods caused by the failure of small dams are usually considered to be an acceptable risk. However, the downstream risk can be a high if the outburst flood changes into a debris flow or mudflow. Volume of the triggered debris flow can 10 to 50 times larger than the initial runoff or mobilized mass at the source area (Hussin et al. 2012).

The frequency of small dam failures is higher than that for large dams. This is mainly because of poor design, poor quality construction, and deterioration due to lack of proper maintenance (FEMA 1987; Pisaniello et al. 2006). In addition, deterioration due to old age is also a factor contributing to dam failure. This study considers the largely unrecognized and specific hazard associated with debris flows triggered by sudden failure of small earth dams. The creation of debris flows as opposed to flooding has rarely been considered as a failure consequence in the downstream dam risk assessment process, and it is not explicitly noted in dam safety guidelines (Table 1.1). A debris flow hazard can change the dam classification in terms consequence of dam failure from low to high. The dam classification is used for management, prioritization, and decision making (CDA 2007).
Table 1.1 Dam classification (CDA 2007)

<table>
<thead>
<tr>
<th>Dam class</th>
<th>Population at risk</th>
<th>Loss of life</th>
<th>Environmental and cultural values</th>
<th>Infrastructures and economics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>None</td>
<td>0</td>
<td>Minimal short-term loss</td>
<td>Low economic losses; area contains limited infrastructure or services</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>no long-term loss</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>No significant loss or deterioration of fish or wildlife habitat</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Loss of marginal habitat only</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Restoration or compensation in kind highly possible</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Temporary only</td>
<td>Unspecified</td>
<td>Significant loss or deterioration of important fish or wildlife habitat</td>
<td>Losses to recreational facilities, seasonal workplaces, and in frequently used transportation routes</td>
</tr>
<tr>
<td></td>
<td>Permanent</td>
<td>10 or fewer</td>
<td>Restoration or compensation in kind highly possible</td>
<td>High economic losses affecting infrastructure, public transportation, and commercial facilities</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Significant loss or deterioration of critical fish or wildlife habitat</td>
<td>Very high economic losses affecting important infrastructure or services (e.g., highway, industrial facility, storage facilities for dangerous substances)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Restoration or compensation in kind highly possible but impractical</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Permanent</td>
<td>100 or fewer</td>
<td>Major loss of critical fish or wildlife habitat</td>
<td>Extreme losses affecting critical infrastructure or services (e.g., hospital, major industrial complex, major storage facilities for dangerous substances)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Restoration or compensation in kind highly possible but impractical</td>
<td></td>
</tr>
</tbody>
</table>
1.1 Objectives

The main objective of this thesis is to develop a method to identify conditions where debris flow initiation may occur if a small earth dam breaches. The method makes use of readily available data such as geological and topographical maps; GIS shape files, and Google Earth. To achieve this goal, the following tasks were performed.

- Determine practical and suitable methods to predict dam breach peak outflow and hydrograph based on limited data available for dams.
- Determine the factors that affect the triggering of debris flows.
- Collect data on selected dams, project and clip geologic maps and Digital Elevation Model and extract data from them to apply in numerical equations.
- Apply numerical equations and verify results using published reports and photos.
- Perform sensitivity analysis to estimate the percent contribution of input variables to the variance of model output.

The thesis has six chapters. Chapter 1 is the introduction. Stream flow is one factor that can trigger a debris flow. Therefore, Chapter 2 presents a method based on empirical equations to predict a peak outflow and a breach hydrograph for small earth dams based on limited available data about dams including the dam height (online dam database) and reservoir area (based on Google Earth measurement). In Chapter 3, selected thresholds for channelized debris flow initiation are introduced and a GIS-applicable relationship is suggested. In addition, the methodology developed for assessment of debris flow initiation caused by small earth dam failures is explained. In Chapter 4 and 5, the Testalinden dam and the Glanzier dam case studies are presented. Chapter 6 presents the conclusions and recommendations.


Chapter 2 Earth Dams

2.1 Earth dams

Earth dams are classified as one of three types: simple, core, and diaphragm. The simple embankment type consists of reasonably uniform material throughout. This type of dam is also referred to as a homogenous embankment dam. Core embankments have a central zone of carefully chosen material, which is less pervious than the rest of the dam. This dam is also referred to as a zoned embankment dam. Diaphragm type dams incorporate a relatively thin section of concrete, steel, or wood – sometimes referred to as cut off wall – in the central portion of the embankment, which forms a barrier to the flow of water percolating through the dam (Stephens 2010; Stone 2003). Figure 2.1 presents three types of earth dams.

![Figure 2.1 Earth dam types](image)

2.1.1 Definition of small earth dams

The definition of a small dam varies worldwide. However, it is based on the height of the dam and the storage capacity of the reservoir (Grapel and Mitchelmore 2009; Mufute et al. 2008). For instance, the International Commission on Large Dams (ICOLD) defines a small dam as a structure that does not qualify as large dam. A large dam is any dam with a height greater than
15 m. Dams that are between 10 to 15 m high are considered a large dam if they are more than 500 m long, retain more than 1,000,000 m$^3$ of water at maximum operating level, or have a flood discharge greater than 2,000 m$^3$/s (ICOLD 1997). Another definition of interest is the definition of a dam in the Canadian Dam Association (CDA) dam safety guidelines (CDA 2007). The CDA defines dam as “a barrier which is constructed for the retention of water, water containing any other substance, fluid waste, or tailing, provided the barrier is capable of impounding at least 30,000 m$^3$ of liquid and is at least 2.5 high”. The definition may be expanded to include dams less than 2.5 m high or with an impoundment capacity less than 30,000 m$^3$ if the consequence of dam operation or failure is likely to be unacceptable to the public, such as:

- dams with erodible foundation that, if breached, could lower the reservoir more than 2.5 m, or
- dams containing contaminated substances.

Therefore, based on these two definitions by two fundamental organizations of dam safety, there is no specific definition for small dams. The definition depends on a dam structure and reservoir location and capacity and consequence of failures. The British Columbia dam safety guideline classifies dams into two groups based on the height of dam. A small dam is a dam that is lower than 9 m high (MFLNRO 2012a).

### 2.1.2 Modes of earth dam failure

Embarkment dam failures are classified into several categories such as: flood overtopping, gate-spillway failure, piping, slope stability, breaching, and earthquake-induced instability (Foster et al. 2000). The two main modes of earth dam failure are piping and overtopping. Piping occurs when seepage builds a tunnel or pipe through an embankment and in extreme cases can lead to undermining and collapse of the dam. Overtopping occurs when the level of a reservoir exceeds the capacity or height of the dam. Piping or overtopping can both lead to breaching of the dam (Figure 2.2). Breaching occurs when a section of the embankment is eroded away, and a gap becomes visible in a dam (MFLNRO 2012a; Umaru et al. 2010).
2.1.3 Small dam issues

Small dams are used for irrigation, municipal water supplies, or storm water management. As consequence, they tend to be close to the service population in comparison with large dams that are typically used for hydroelectric generation. Because of cost constraints, typically less time and effort are spent for engineering and construction of small dams. This may lead to substandard design and poor construction quality. Local contractors are often hired to construct, repair or maintain small dams. These contractors have limited resources and sometimes use farm dam practices during construction with resulting poor long-term performance.

Some dam owners believe that old dams tend to get better with age (Grapel and Mitchelmore 2009). Therefore, some dam owners believe there is little need for regular maintenance or attention. The limited resources of a small dam owner can lead to lack of monitoring and rehabilitation for dams. The minimal awareness of hazards associated with small dams could result in the downstream consequences of a dam failure not being fully recognized (Grapel and Mitchelmore 2009). Moreover, owners of small dams rarely record and maintain detailed dam surveillance and monitoring reports. Reports that may be available typically do not record all details recommended by the BC Dam Safety Guideline (MFLNRO 2012a) such as upstream
slope, downstream slope, crest length, and spillway depth and width (Noseworthy 2012). Estimating the probability of failure of these dams has high uncertainty although, it might be quite high given that 8 dams have already failed in the Okanagan valley since 1921 (Tannant and Skermer 2011).

The issue investigated in this thesis is the debris flow hazard that can be initiated by the outburst of a dam located high above a valley.

The sudden failure of a dam on a creek, whether of a natural landslide dam or an artificial dam, can result in a large discharge of water downstream. The surge of escaping water can easily turn into a mudflow or debris flow by incorporating inorganic and organic debris from the creek channel floor and sides into the floodwaters. Debris flows are a complex group of gravity induced rapid mass movements intermediate between landslides and water flooding. Debris flows include a variety of grain sizes from boulders to clay mixed with varying amounts of water. Mudflows contain mostly fine-grained material, such as sand, silt, and clay, mixed with water. Mud flows and debris flows triggered by such events can travel for many tens of kilometres downstream (Skermer and VanDine 2005; Clague et al. 1985) and damage houses and structures located downstream from the dam.

Table 2.1 lists the historical record of dam failures in the Okanagan Valley. The Testalinden Dam failure, which is the most recent dam failure, caused a massive debris flow and significant damage; this dam failure and debris flow is the main case study investigated in this thesis. The methodology developed for predicting debris or mudflow initiation caused by dam failure and results of analysis are given in Chapters 3 and 4.
Table 2.1 Dam failures in the Okanagan valley (Tannant and Skermer 2011)

<table>
<thead>
<tr>
<th>Dam Name</th>
<th>Place</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ellis Creek No. 3 Dam</td>
<td>Penticton</td>
<td>May 19, 1921</td>
</tr>
<tr>
<td>Shuttle worth Creek Allendale Dam</td>
<td>Okanagan Falls</td>
<td>May 15, 1928</td>
</tr>
<tr>
<td>Shuttle Worth Creek Campbell Meadows Dam</td>
<td>Okanagan Falls</td>
<td>May 15, 1936</td>
</tr>
<tr>
<td>Wagner Dam Loup, Loup Creek</td>
<td>Malott</td>
<td>April 19, 1938</td>
</tr>
<tr>
<td>Ellis Creek No. 4 Dam</td>
<td>Penticton</td>
<td>May 12, 1941</td>
</tr>
<tr>
<td>Ellis Creek No. 4 Dam</td>
<td>Penticton</td>
<td>May 23, 1942</td>
</tr>
<tr>
<td>Shuttle Worth Creek Campbell Meadows Dam</td>
<td>Okanagan Falls</td>
<td>May 27, 1947</td>
</tr>
<tr>
<td>Testalinden Dam</td>
<td>South of Oliver</td>
<td>June 13, 2010</td>
</tr>
</tbody>
</table>

2.2 Dams in BC

The MFLNRO (2012) provides a database about registered dams for the province in Google Earth format. According to the latest version of this database, there are 1692 dams in British Columbia. Figure 2.1 shows the dams in BC classified based on construction material.

![Figure 2.3 Dam types in BC](image)

There are 1389 earth dams in BC and 93% of these are classified as small dams. Dam owners are responsible for the operation, maintenance, and surveillance of these dams. Given that most
small dams are privately owned, most dam owners have limited resources for adequate engineering inspection, repair, and dam safety management programs (Grapel and Mitchelmore 2009). Therefore, these factors can increase the probability of dam failure.

2.2.1 Study area and case study

The Okanagan valley has a dry and sunny climate. Much of the land development is localized in the valley bottom. There are 204 earth dams in the Okanagan valley region. Many of these dams are located high above the valley floor. If one of these dam breaches because of geomorphic characteristics of the region, dam breach outflow will intensify the debris flow hazard and create the risk of damage for infrastructure constructed on the alluvial fans at the bottom of the valley. Figure 2.4 presents a map of earth dam locations in the Okanagan region. Dams are classified into the three groups: small dams, big dams and others. ‘Other dam’ means that the information about the dam height is not available in the database.
Figure 2.4 Earth dams in the Okanagan Region
2.3 Dam safety evaluation

There are two viewpoints to dam safety evaluation; the conventional safety oriented or standards based approach, and the risk-based approach (Rettermeir et al. 2001; Bowels et al. 1997; Harrald et al. 2004).

2.3.1 Standards-based approach

Standard-based safety evaluation system is based on design, construction, inspection and maintenance guidelines or standards usually set by the responsible authority mandated to oversee dam issues in a country or part of country (Bowels et al. 1997). The system is based on monitoring, surveillance, inspection, and maintenance programs.

2.3.2 Risk-based approach

The term “risk-based approach” involves assessment of acceptable safety defined using information provided from a risk assessment and other decision inputs. Risk assessment is a systematic process where experienced dam engineering professionals provide decision makers with estimates of the risks and associated uncertainties of system responses, outcomes, and consequence. Figure 2.5 presents the risk assessment process for earth dams (Mufute et al. 2008).
This thesis focuses on debris flow hazards triggered by the breaching a small earth dam. This mechanism is typically not considered as a hazard in dam failure risk assessments.

### 2.4 Dam failure consequences

Most dam safety guidelines in use around the world suggest examination of the following dam failure consequences: life safety, economic impacts, and environmental and cultural loses (CDA 2007; Water Act 2011; Alberta Environmental Protection 1991).

#### 2.4.1 Life safety

It is very difficult to develop consistent estimation of loss of life, as the potential for loss of life depends on many highly uncertain and variable factors. To quantify life safety in some approaches, the population at risk in the inundated area are verified as people exposed to the hazard (CDA 2007).
2.4.2 Economic impacts

Economics and social losses are defined as damage to third-party property, facilities, other utilities and infrastructure. The damage to the dam owner's property is usually excluded from consideration. It seems reasonable to leave dam owners to decide how to deal with those losses (CDA 2007).

2.4.3 Environmental and cultural losses

The importance of environmental losses should be evaluated in terms of whether restoration of the environment is feasible and how long it would take. The magnitude and duration of impact should be considered in the loss evaluation. Social impacts such as damage to irreplaceable historic and cultural features, which cannot be evaluated in economic terms, should be considered on a site-specific basis.

Dams are usually classified based on consequences of failure to provide guidelines for the required monitoring and the care expected of dam owners and designers. Table 1.1 presents the most recent dam classification in terms of failure consequences used as a main reference for provincial guidelines.

The Testalinden Dam, which is discussed in more detail in Chapter 4 was classified a “low” dam class in terms of consequence of failure. However, the debris flow triggered by the Testalinden dam failure caused financial losses of approximately 20 million dollars and endangered human life downstream of the dam.

2.5 Dam breach peak outflow and hydrograph

The main factor triggering a debris flow is water. A source of water can be rainfall, snowmelt runoff, or dam outburst. In most of the research regarding debris flow initiation, the source of water is rainfall (Godt and Baum 2010; Blahut et al. 2009) and dam breaching outflow is rarely considered in the debris flow hazard assessment. The source of water in this study is dam breach outflow. Breach peak outflow and hydrograph can be estimated in several ways: 1) comparative analysis that compares the dam of interest with historical failures of dams of similar size, material and storage volume, 2) regression equations developed from historical dam failures that
correlate breach size and development time with dam and reservoir characteristics, 3) numerical model based on modified weir equations or sediment transport equations, and 4) dam flood routing software (Gee 2010). Method 2 is used in this study. The historical data are too limited to apply the Method 1. Methods 3 and 4 need detailed data (dam structure and creek cross section) that cannot be extracted from maps and low-resolution digital elevation models.

2.5.1 Empirical equations for peak outflow, \( Q_p \)

Empirical relationships obtained using data from previous dam failures can be used to estimate the peak outflow through a dam breach. Empirical regression equations were developed as a function of dam geometric parameters such as dam height and/ or reservoir storage volume to predict peak outflow (Thornton et al. 2011; Wu 2011). Since 1977, different individual regression expressions have been recommended to predict peak outflow. Table 2.2 presents a summary of selected regression equations for estimation of peak outflow through a dam breach. The variables in the relations are \( Q_p = \) peak outflow \( (m^3/s) \), \( h = \) height of water behind dam, \( h_d = \) dam height \( (m) \), \( H_w = \) height of water above the breach invert at failure \( (m) \), \( V_w = \) volume of water behind the dam at failure \( (m^3) \), \( V = \) volume of water behind dam \( (m^3) \).
Table 2.2 Selected empirical equations of peak outflow through a dam breach

<table>
<thead>
<tr>
<th>Author</th>
<th>Equation</th>
<th>Number of case studies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kirkpatrick (1977)</td>
<td>( Q_p = 1.268(H_w + 0.3)^{2.5} )</td>
<td>13</td>
</tr>
<tr>
<td>Soil Conservation Service (1981)</td>
<td>( Q_p = 16.6(H_w)^{1.85} )</td>
<td>13</td>
</tr>
<tr>
<td>U. S Bureau of Reclamation (1982)</td>
<td>( Q_p = 19.1(H_w)^{1.85} )</td>
<td>21</td>
</tr>
<tr>
<td>Hagen (1982)</td>
<td>( Q_p = 1.205(V_w H_w)^{0.48} )</td>
<td>6</td>
</tr>
<tr>
<td>Singh &amp; Snorason (1982)</td>
<td>( Q_p = 13.4(h_d)^{1.89} )</td>
<td>8 (simulated)</td>
</tr>
<tr>
<td>Singh &amp; Snorason (1984)</td>
<td>( Q_p = 1.776(V)^{0.47} )</td>
<td>8 (simulated)</td>
</tr>
<tr>
<td>MacDonald &amp; Langridge-Monopolis (1984)</td>
<td>( Q_p = 1.154(V_w H_w)^{0.412} )</td>
<td>23</td>
</tr>
<tr>
<td>MacDonald &amp; Langridge-Monopolis (1984)</td>
<td>( Q_p = 3.85(V_w H_w)^{0.412} )</td>
<td>23</td>
</tr>
<tr>
<td>Costa (1985)</td>
<td>( Q_p = 0.763(V_w H_w)^{0.42} )</td>
<td>31</td>
</tr>
<tr>
<td>Evans (1986)</td>
<td>( Q_p = 0.72(H_w)^{0.53} )</td>
<td>29</td>
</tr>
<tr>
<td>Froehlich (1995)</td>
<td>( Q_p = 0.607(V_w^{0.295} H_w^{1.24}) )</td>
<td>22</td>
</tr>
<tr>
<td>Pierce et al. (2010)</td>
<td>( Q_p = 0.0176(V h)^{0.606} )</td>
<td>87</td>
</tr>
<tr>
<td>Pierce et al. (2010)</td>
<td>( Q_p = 0.038(V^{0.475} h^{1.09}) )</td>
<td>87</td>
</tr>
</tbody>
</table>

Equation 1 (Froehlich 1995) is believed to be the best empirical equation for predicting peak outflow from earth dams as noted in the uncertainty analysis conducted by Wahl (2004, 2010). This equation is used later to predict the peak outflow from two different dams in the case studies.

\[ Q_p = 0.607(V_w^{0.295} H_w^{1.24}) \]

2.5.2 Dam breach hydrograph

Major dam flood routing methods such as BREACH and HEC-RAS includes dam breach modeling for estimating consequence of dam failure. The dam breach assessment involves determining peak discharge outflow and a discharge hydrograph (US Army Corps of Engineers 1977; Singh and Scarlatos 1987). The discharge hydrograph provides the rate at which the
volume of water is released from the breached dam. Methods for developing a discharge hydrograph include a simple triangular approximation and physically-based models.

One group of physically-based models such as BREACH (Fread 1988) simulate breach initiation as a tractive-force erosion problem. Wu (2011) noted these models are often not consistent with the erosion mechanics observed in laboratory testing and documented case studies of dam failures. The information available for the small dams in the Okanagan region does not include dam material properties (friction angle) and crest length, which are needed as an input to models such as BREACH. Therefore, these methods are not used in this study.

Another group of physically-based models is based on modified weir equations such as HEC-RAS, (US Army Corps of Engineers 2010). These require the geometry of the breach and the breach formation time as input. The output will give the breach enlargement as a function of time and the outflow hydrograph (US Army Corps of Engineers 2010). The required input parameters should be found from a comparative method (Wahl 1998) or from empirical regression equations, such as (Froehlich 1995). The comparative method is difficult to use because historical data of time of failure for small earth dams is limited. Moreover, small variations in breach parameters (time of failure, initial breach width, final breach width) cause large changes in predicted peak outflows for dams with small reservoirs (Wahl 1998).

Therefore, a simple triangulation approximation is chosen to create the outburst hydrograph in this thesis. This method has the advantage that the predicted peak outflow is independent from the time of failure. A triangular-shaped hydrograph is constructed using the peak outflow (Equation 1), time of failure (Equation 2) and the reservoir volume. The breach discharge is assumed to increase linearly from zero at the time of breach initiation to the value of the peak outflow $Q_p$, at the time of the breach, $t_f$. From the peak, the breach discharge is then assumed to linearly decrease to zero. The time at which the discharge reaches zero is set such that the total area under the hydrograph is equal to the reservoir volume at the time of the breach initiation; $V_w$. Figure 2.6 shows a typical triangular hydrograph.
2.5.3 Empirical equations for time of failure, $t_f$

The time of failure, $t_f$ (in hours) is the time it takes to reach the peak outflow through a dam breach from the moment of initiation. This parameter can be used to help construct a dam breach hydrograph. Empirical equations are often used to estimate the time of failure. Some of these equations are listed in Table 2.3. In addition to the volume of water in the reservoir and the dam height, the average breach width, $B$ (m), is sometimes used in these equations.
Table 2.3 Selected empirical equations for estimating time of dam failure

<table>
<thead>
<tr>
<th>Author</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Singh &amp; Snorrason (1982)</td>
<td>$0.25 \ H_w &lt; t_f &lt; 1.0 \ H_w$</td>
</tr>
<tr>
<td>Macdonald &amp; Langridge (1984)</td>
<td>$0.25 \ H_w &lt; t_f &lt; 1.0 \ H_w$</td>
</tr>
<tr>
<td>FERC (1987)</td>
<td>$0.1 \ H_w &lt; t_f &lt; 0.5 \ H_w$</td>
</tr>
<tr>
<td>Froehlich (1995)</td>
<td>$t_f = 0.00254 V_w^{0.53} H_w^{0.09}$</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette (1990)</td>
<td>$t_f = 0.02 H_w + 25$ (erosion resistant)</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette (1990)</td>
<td>$t_f = \frac{B}{4 H_w}$ (erosion resistant)</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette (1990)</td>
<td>$t_f = 0.015 H_w$ (easily erodible material)</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette (1990)</td>
<td>$t_f = \frac{B}{4H_w + 61}$ (easily erodible material)</td>
</tr>
</tbody>
</table>

Wahl’s (2004) uncertainty analysis found that the equation proposed by Froehlich (1995) is best suited for predicting the time of failure for dams (Wahl 2010). Thus Equation 2 is used later to predict failure time for two different dams in the case studies.

$$t_f = 0.00254 V_w^{0.53} H_w^{0.09}$$
Chapter 3 Debris Flow Initiation

Chapter 3 describes the selected numerical relations that were developed to identify channelized debris flow initiation thresholds. The methods used to couple these relations to data typically available in Geographic Information Systems (GIS) are discussed.

3.1 Debris flows and mud flows

Debris flows include a range of grain sizes from boulders to clay mixed with varying amounts of water. Because of the high velocity and inertia of many debris flows, they represent significant geological hazards. Sediment concentration in debris flows ranges from 70-90% by weight (47 to 77% by volume) and both water and solids move together as a unit at the same velocity (Ritter et al. 2002; Rickenmann 1999). Although numerous ideas exist concerning how debris flows are mobilized, most researchers feel that they begin either as debris-laden slurry that erodes its own channel and thus increases its sediment concentration, or as a shallow landslide that provides a high concentration of unconsolidated debris and mobilizes as a flow when runoff is mixed with the debris. The abundant moisture necessary for debris flows is commonly provided by intense rainfall, rapid snowmelt, or dam rupture. Debris flows typically start in small drainage basins where slopes are steep, where runoff can be concentrated, and where sediment supply is likely to be large (Highland and Bobrowsky 2008; Hofmeister and Miller 2003). Therefore, debris flow can occur if these three factors exist together: adequate runoff, enough sediment supply, and steep area. In this thesis, to derive required input data, 1) dam breach peak outflow and hydrograph data are used to estimate runoff, 2) surficial geology maps and Google Earth are analysed to determine sediment supplies, and 3) digital elevation models are used to determine the gradient of the creeks.

The causes of debris flow initiation can be classified into three types (Takahashi 2007).

1) Flowing water erodes the deposits in the gully bed and the concentration of solids in the surface water flow increases until it becomes a debris flow.

2) A landslide transforms into a debris flow while in motion by the effects of stored water in the displaced soil.
3) The sudden failure of a dam (natural or man-made) can cause a debris flow.

### 3.1.1 Development of debris flow due to erosion of a gully bed

On a steep gully bed or channel, sediment accumulates gradually because of the supply from the sidewalls as rock-falls and landslips. The gully is a path for intermittent surface water flow. During severe rainfall, or rapid melting of snow or ice, the resulting water flow can erode the channel and trigger debris flow hazard (Prochaska et al. 2008; Yu 2011).

### 3.1.2 Landslide-induced debris flow

Two types of landslides can occur during a severe rainstorm; a shallow landslide of about 1 m thick and a deep-seated landslide. A shallow landslide usually occurs with the strongest rainfall intensity, whereas, the deep-seated one often occurs after the rainfall ends. The shallow landslide contains plenty of water in itself, and it is helped by a high runoff discharge. For a severe rainfall, the sliding mass of soil can be easily transformed into a debris flow almost from the instant of motion initiation (Coe et al. 2007; D'Agostino and Marchi 2010).

A deep-seated landslide needs a comparatively long time before the ground water level to rise enough to make the earth block unstable. Therefore, it often occurs later than the time of the strongest rainfall and by that time flood runoff around the landslide may already have been reduced. Hence, the procedure for the transformation into debris flow would be completely different from the shallow landslide (Scheidl 2009; Takahashi 2007).

### 3.1.3 Debris flow induced by the collapse of a dam

A landslide mass can slide into a river channel and block the flow. This called a natural dam or landslide dam. The collapse of a natural dam or a man-made dam can cause a debris flow (Takahashi 2007; Clague et al. 1985; Chen et al. 2004). Debris flow initiation caused by outflow from a dam breach that erodes the channel bedding is a mechanism similar to that presented in Section 3.1.1. Note that the routing and deposition of the debris flow materials are not in the scope of the thesis.
3.2 Debris flow initiation thresholds

Various researchers have studied debris flow mechanisms: initiation, routing, and deposition. Different thresholds have been developed for debris flow initiation such as rainfall intensity or Melton ratio (Godt and Baum 2010; Welsh and Davies 2011; Blahut et al. 2009; Wilford et al. 2004). In this thesis, channelized debris flow initiation caused by dam-breached outflow is investigated. Other researchers have not extensively studied this mechanism. The thresholds that were developed by Takahashi (2007), Berti and Simoni (2003), and Yu (2011) are used. The common point in these studies is that they developed a threshold that considered all the main factors that cause debris flows: slope, stream flow, and sediments in the creek. A summary of these three studies is presented in the following sections.

3.2.1 Takahashi (2007)

Takahashi (2007) developed a numerical method to determine criteria for debris flow initiation. An infinitely long, uniform and void rich sediment layer saturated with water is considered (Figure 3.1). The sediment has a thickness $D$ and a slope $\Theta$. On the surface, water of depth $h_0$ is flowing. The hypothetical shearing stress $\tau$ that acts to drag the sediment block downstream and the resisting stress, $\tau_r$, are assumed to distribute in straight lines (Figure 3.1). A debris flow will be generated if the shear stress imposed by the sediment and flowing water exceeds the shear strength of the sediment layer on the failure plane (Takhashi 2007).
The shear stress at depth, \( a \), from the surface of the sediment layer is calculated as follows:

\[
\tau = g \sin \theta \left[ C_s (\sigma - \rho) a + \rho (a + h_0) \right]
\]

In addition, the shear strength (resisting stress) at depth, \( a \), is assumed to be governed by the Mohr-Coulomb failure criterion:

\[
\tau_r = g \cos \theta \{ C_s (\sigma - \rho) a \} \tan \varphi + c
\]

The parameters used in the two equations are:
- \( \tau \) = shear stress
- \( \tau_r \) = shear strength
- \( g \) = acceleration due to gravity
- \( \theta \) = creek or gully gradient
- \( C_s \) = concentration of solids when packed
- \( \sigma \) = particle density
- \( \rho \) = fluid density
- \( h_0 \) = height of water
- \( \varphi \) = internal friction angle of sediment layer
- \( c \) = cohesive strength of sediment layer.
Based on the above equations and results of experiments done by Takahashi (2007), a criterion for debris flow initiation was established. Debris flows are predicted to occur in a gully bed that is steeper than the critical gradient, $\theta_1$ when surface water flow meets the condition: $q_0 \geq 2$.

These two conditions that must be satisfied for a debris flow to occur are:

$$q_0 = \frac{q_0}{g^{1/2} d_P^{3/2}} > 2$$

$$\theta > \theta_1 = \arctan\left(\frac{c_s(\sigma-\rho)}{c_s(\sigma-\rho)+\rho(1+k^{-1})}\tan\varphi\right)$$

where $q_0$ is the water discharge per unit width flowing down the channel, $k$ is the lateral transmission coefficient of saturated seepage flow, and $d_P$ is the particle diameter of the sediment.

### 3.2.2 Berti and Simoni (2003)

In most methods for determining the threshold for debris flow initiation, the particle size is an important input parameter (Armanini and Gregoretti 2000; Takhashi 2007). However, in practical terms, stream channel sediments can have a wide range of particle sizes with grain-size distribution ranging over four orders of magnitude (Berti and Simoni 2003). The use of models based on particle size to estimate debris flow initiation thresholds, such as those by Armanini and Gregoretti (2000) and Takhashi (2007), relies on the selected value for the particle size, which can be problematic in estimating.

To overcome this problem, Berti and Simoni (2003) developed a threshold for debris flow initiation that is independent of particle size. This criterion was established by their field observations of debris flows in the Acquabona debris flow catchment. Equation 7 developed by Berti and Simon (2003) is used to calculate the safety factor that represents the ratio between shear strength and acting shear stress on a slope-parallel plane located at a depth $Z$:

$$FS = \frac{c+\left[y_{sat} Z \cos^2 \theta - y_w Z \cos \theta \cos \beta \cos \varphi \right]}{(y_{sat} Z + y_w h_w) \sin \theta \cos \theta}$$

Where
\( \gamma_{sat} = \) unit weight of saturated sediment

\( \gamma_w = \) unit weight of water

\( \beta = \) direction of groundwater flow vector measured with respect to horizontal

\( h_w = \) height of surface water flow.

Figure 3.2 illustrates the parameters used in Equation 7. When \( FS < 1 \), a debris flow initiates. Three challenges for using Equation 7 are: 1) estimating the height of surface water flow, 2) estimating the direction of the groundwater flow vector, and 3) determining an appropriate depth below the channel surface (\( Z \)) to apply the equation.

![Figure 3.2 Schematic of creek model (modified from Berti and Simoni 2003)](image)

The height of water flowing in the channel \( h_w \) is needed to apply the debris flow initiation equation (Equation 7). To estimate the maximum height of flowing water in the creek channel, Manning’s equation, the peak outflow \( Q_p \), and the continuity equation (Chanson 2004) are used (Equations 8 and 9).

\[
V = \frac{1}{n} R^{2 \frac{1}{2}}
\]

\[
Q_p = VA
\]

Where
\[ V = \text{velocity of water (m/s)} \]

\[ n = \text{Manning's roughness coefficient} \]

\[ R = \text{hydraulic radius of the channel (m)} \]

\[ S = \text{slope of channel (vertical drop divided by horizontal travel distance)} \]

\[ Q_p = \text{peak outflow from dam breach (m}^3/\text{s)} \]

\[ A = \text{wetted area of channel cross-section (m}^2). \]

The shape of the channel cross section must be either measured or assumed in order to determine the wetted area, \( A \) and hydraulic radius, \( R \). By substituting Equation 8 into Equation 9 and assuming the channel cross-section has a trapezoidal shape, the value for \( h_w \) can be solved.

The direction of the groundwater flow in the sediments lining the channel is assumed to be parallel to the channel gradient \( \beta = \theta \). This implies no flow occurs across the failure surface shown in Figure 3.2. This is a simplifying assumption and is likely most valid when the channel sediments are saturated and located above impermeable bedrock.

The depth of debris, \( Z \), at which the factor of safety is calculated using Equation 7 must be estimated. The approach used in this thesis is to relate the peak height of water flowing down the channel to the depth, \( Z \). The assumption is that all sediment down to a depth, \( Z \), corresponding to a \( FS = 1 \) is mobilized into a debris flow by the flowing water at a height, \( h_w \). Furthermore, the resulting debris flow has a sediment solids concentration, \( Sc \) that is typical for debris flows. Observed debris flows have typical sediment concentrations of 47% to 77% by volume of the debris flow, Equation 10 (Ritter et al. 2002; Rickenmann 1999).

\[ Sc = V_s / V_t \]

\( V_s \) is volume of the solids, \( V_t \) is total volume of the debris flow.

The initial void ratio, \( e \), of the channel sediment dictates the concentration of the solids before the sediment is mobilized into a debris flow. The sediment in the channel is assumed to be fully
saturated before the debris flow occurs thus the volume of the water in the solid is equal to the volume of the voids.

\[ e = \frac{V_v}{V_s} \]

By applying phase relations for a soil and Equations 10 and 11 (Holtz et al. 2011), the ratio of the depth of the debris to the height of runoff water per unit width of the channel can be calculated by using Equation 12, for unit length and width of channel \( (V_s \equiv Z) \). The debris flow is assume to be fully saturated, therefore the volume of voids equals the volume of water (Figure 3.3). The total volume of water in the debris flow is the sum of the peak channel flow height (per metre unit length and width of the creek) created by the outburst and the volume of pre-existing water in the creek sediments.

\[ \frac{Z}{h_w} = \frac{Sc}{(1 - Sc - e \cdot Sc)} \]

For silty sand to sandy gravel soil types, the typical void ratio is 0.25 to 0.3 (Terzaghi et al. 1996). These values can be used to estimate the water present in the creek bed sediments. Using Equation 12 and expected sediment concentration for a debris flow when it is triggered (47% to 60%), the ratio \( \frac{Z}{h_w} \) predicted from Equation 12 ranges from 1 to 3.

**Saturated creek sediments**

**Debris flow**

![Diagram](image)

Figure 3.3 Soil phase diagram for creek sediment and resulting debris flow
3.2.3 Yu (2011)

Yu (2011) investigated debris flow initiation in the Jiangija gully in China. Different debris flow initiation thresholds proposed by Chinese researchers are investigated in Yu's study. According to his work, Equation 13 can be used to estimate the maximum diameter, $d_p$ (m), of the mobilized coarse particles based on the unit discharge in the channel ($m^3/m\cdot s$). By using Equation 13, the particle size that can be eroded by the flow of water from a dam outburst can be predicted.

$$d_p = 0.04q_0^{2/3}$$

Yu (2011) stated that ordinary flow in a channel can carry away fine particles, but to form a debris flow, the diameter of the particles mobilized by flowing water should be greater than 20 mm.

3.3 Sources of data for predicting debris flow initiation

To obtain data for predicting debris flow hazards, several common methods are recommended: assessment of previous debris flow events, air photo interpretation, field reconnaissance, and spatial GIS analysis. Interpretation of recorded debris flow events is a common way for evaluating future potential debris flow hazards for a specific area, but this type of data is barely available for our studied area, Okanagan Valley. Field investigation is the other way for stream analysis and it requires surveying a region to evaluate geology, geomorphology, and topography for preparing data for engineering judgement and further analysis, but this process time consuming and costly. Therefore, computerized analysis was chosen for this research. This required obtaining a digital map for geospatial analysis and spatial and structural information about each dam. In the following section, data collecting and analysing methods are explained.

3.3.1 Dam breach hydrograph

In all the equations mentioned above for debris flow initiation, the height of water or unit discharge is an important factor. In this study, the source of water is a predicted dam breach outflow that should be calculated according to the dam database and breach models. The MFLNRO (2012) has a database in Google Earth format for dams in BC. The database contains the name, georeference location, height, crest length, type, category, and function of the dams.
The predicted breach peak outflow and hydrograph can be estimated by analysing the extracted data. Empirical equations are used to predict the peak outflow and the time of failure using basic data available for dams in the Okanagan region.

3.3.2 Creek gradient and cross-section

The creek gradient is one of the factors that must be known when investigating debris flow initiation. A digital elevation model (DEM) is used for determining the gradient of creeks in this study. Steepness controls the shear stress needed to start the debris flow. The gradient of a stream channel is derived from DEM data by applying 3D analyst and spatial analysis features of ArcGIS software.

Creek gradient ranges from 5° to 22° for debris flow initiation in recorded events. The variation in slope gradient for debris flow initiation is related to different amounts of water and terrain geology in the events (Jakob and Hungr 2005; Fannin and Rollerson 1992; VanDine 1984).

A DEM represents the terrain surfaces as a regular lattice of point elevations. A grid can be considered a tessellation using square tiles with a point elevation at the centre of each square, as shown in Figure 3.4. Gridded DEMs may be calculated directly by stereoscopic interpretation of data collected by airborne and satellite sensors. The traditional source of this data is aerial photography (Wilson and Gallant 2000).

Figure 3.4 Digital elevation model
Existing Geographic Information Systems (GIS) can be used as part of the debris flow hazard assessment procedure. For example, BC has a publically available online dam database that can be access using Google Earth (MFLNRO 2012b). In addition, DEMs are available from Geo Base (2012). GIS can be used to identify the areas that have potential for debris flow hazards, and in dam failure scenarios, GIS can help to delineate the movement of dam outburst flow and determine the inundated areas (Wang and Ferris 2010; Xiaobo and Peng 2007; Wang et al. 2006). In this research, GIS is used to prepare data required for analysing debris flow initiation and spatial analysis.

3.3.3 Surficial geology and creek bed sediments

Abundance of unconsolidated sediment is essential for debris flow occurrence. The amount and type of loose material are important material conditions because they determine whether a debris flow could initiate and have an impact on quantity of water required to trigger a debris flow (Wie et al. 2008). In this research, the surficial geology map of the studied area, engineering reports, and photos are used to evaluate the surficial geology conditions.

There should be abundance of loose sediment in the creek channel that can be eroded and mobilized by the runoff to initiate a debris flow. A surficial geology map is used to derive sediment information along a creek. After determining sediment type in different sections of a creek, the mechanical properties of the soil (average particle diameter, cohesion, and friction angle) are estimated by using data suggested in textbooks. These properties are used in the debris flow initiation equations (Equations 5, 6, and 7).

3.4 Tools for evaluation of debris flow hazard

In order to predict the debris flow hazard associated with dam failures in an area, the conditions and factors that trigger instability must be identified and their relative contributions to failure estimated. There are several methods to evaluate debris flow hazard: map analysis, aerial reconnaissance, field reconnaissance, and computerized terrain analysis (Hofmeister and Miller 2003).
3.4.1 Map analysis

Map analysis is usually one of the first steps in landslide or debris flow investigation. Bedrock and surficial geology, topography, and soil maps are required for investigation. An experienced person can obtain a general idea of debris flow susceptibility from such maps and using knowledge of geologic materials (Giraud 2005).

3.4.2 Aerial reconnaissance

Analysis of aerial photography and laser scanning provides a three dimensional overview of the terrain and indicates human activities as well as geologic information for a trained person which can assist him in identifying debris flows and landslides (Staley et al. 2011; Tian and Zhang 2009).

3.4.3 Field reconnaissance

Many of signs of slope movement cannot be recognized on maps or photographs. Indeed, if an area is heavily forested or has been urbanized, even major features may not be clear. Therefore, field investigation is always essential to verify or detect landslide or debris flow features, and to evaluate critically the potential instability of vulnerable slopes (Davies et al. 2005).

3.4.4 Computerized debris flow analysis

In recent years, computer modeling of debris flow has been used to analyse debris deposition in the fan, debris flow routing, and spatial analysis to create susceptibility maps. Very promising methods have been developed to use DEM to evaluate areas for debris flow hazards (Blais-Stevens et al. 2011; Hsu et al. 2010; Chau and Lo 2004; Mikos et al. 2006; Wang et al. 2006).

3.4.5 Methodology for assessing debris flow initiation

A methodology is developed to investigate debris flow initiation in terms of dam breach hydrograph modeling and using GIS data for spatial analysis. The dam breach hydrograph modeling and computerized debris flow analysis are integrated to obtain results. Figure 3.5 summarizes a proposed methodology for quickly screening and assessing dams that have a potential to initiate a debris flow if a dam were to fail. Three factors are required to assess debris
flow initiation: 1) the height of water or unit discharge in the creek, 2) the depth and shear strength (cohesion and friction angle) of sediment in the creek, and 3) the creek gradient.

To estimate the height of water in a creek downstream of a dam breach, information is needed on the height of the dam and the reservoir volume at the time of the breach. Data for dams can be found from an online dam database, dam monitoring reports, and Google Earth images. Froehlich’s (1995) equations (Equations 1 and 2) were found to be suitable and practical to predict the peak breach outflow and time of failure. From these values, a breach hydrograph can be estimated. The maximum height of water flowing down the creek is then estimated using Manning’s equations and estimates of the creek cross-section geometry.

To estimate the shear strength of sediments lining a creek channel, data concerning the soil types along different sections or reaches of the creek are extracted from surficial geology maps. Typically, cohesion is assumed to be zero for creek bed sediments and the friction angle for the sediments can be simply estimated using typical values for different soil types.

The creek channel gradient or slope can be found from spatial analysis of DEMs or using Google Earth to create a channel longitudinal profile. Using the creek profile and maps of soil types, the creek is broken into reaches with similar characteristics for further analysis to estimate the potential for debris flow initiation. This methodology is valid up to the point where a debris flow initiates in a specific reach of the creek after which, water flow may change to a debris flow. Therefore, the methodology does not apply to reaches below the first reach in which a debris flow is predicted to initiate.
Figure 3.5 Methodology to assess debris flow initiation caused by dam failure
Chapter 4 Testalinden Dam Case Study

On June 13, 2010, a small earth dam failed on the upper watershed of Testalinden Creek, located south of the town of Oliver, BC (Figure 4.1). The water released by the breach in the dam initiated a debris flow that flowed down to the Testalinden Creek fan. The debris flow resulted in extensive property damage on the Testalinden Creek fan (Everest 2010; Culbert 2010). Testalinden Dam is the main case study in this research.

Figure 4.1 Topographic map of Testalinden Creek (modified from Atlas of Canada 2012)

Once the debris flow reached the lower gradient fan, deposition began forming a long asymmetrical triangle of sediment that buried the original channel and created new one that is approximately 1.4 km long (Figure 4.2). The overall area on the fan affected was approximately 23.6 ha. According to the test pit results, and lidar surveys conducted on the fan for the Regional District Central Okanagan, the estimated volume of the debris and sediment is 240,000 to 260,000 m³ (Higman et al. 2011).
The debris flow destroyed or badly damaged five homes and caused significant damage to crops and farm equipment, covered 200 m of Highway 97 and blocked several secondary roads. The Testalinden Dam failure made the BC government set a reemphasis and priority on dam safety. The BC Deputy Solicitor General made a number of dam safety recommendations and BC dam safety officers were busy for two years implementing these recommendations. Some these actions were performing a rapid dam assessment, revising the BC Dam Safety Regulation, updating of the dam registry, creating greater access to information by dam owners, providing local government and the public with tools such as iMap and Google Earth (BC Dam Safety Program 2011; 2012).
The Testalinden Dam was rated as a “low” consequence dam on the BC Dam Safety Regulation Downstream Consequence Classification Schedule. A “low” classification describes a low potential for loss of life, low economical loss and cost, and low loss or significant deterioration of regionally important environmental or cultural habitats that may recover overtime without restoration. A “low” rating would imply the need for a dam audit every 10 years. In hindsight, it appears that “low” rating for the Testalinden Dam was likely inappropriate (Morhart 2010).

4.1 Testalinden Lake and Dam

The Testalinden Dam was located on the upper Testalinden Creek on Mount Kobau (Figure 4.1). It sits within the South Okanagan Grasslands Protected Area. The reservoir is locally known as Testalinden Lake, Kobau Lake and Kobau Reservoir (Figure 4.3). The dam was constructed in 1937 for the purpose of water storage for irrigation in the drier months of the year. Seasonal snowpack is the primary source of water stored in the reservoir (Morhart 2010). The reservoir capacity at the point of overtopping and its surface area are about 20,000 m³ and 14,400 m² respectively (Tannant and Skermer 2011). It was a simple earth dam with a maximum height of about 2.5 m, and no engineering design and supervision occurred during construction. The current owner has been responsible for this dam’s safety since 1985. There was sufficient evidence to see a consistent pattern of concerns and warnings about the condition of the dam dating back to the 1960s. There is no sign that actions had been taken to fix deficiencies in the dam structure that had persisted for decades (Morhart 2010).
4.2 Testalinden Creek

Testalinden Creek is 8.8 km long. The creek starts at Testalinden Lake at an approximate elevation of 1810 m near the top of the watershed. The uppermost elevations of the Testalinden Creek watershed are at an approximate elevation of 1850 m. Testalinden Creek flows eastward into the Okanagan River at an approximation elevation of 285 m. The drainage area of Testalinden Creek is 12.65 km².

A large alluvial fan is located where the creek exits a narrow bedrock canyon and encounters the wide Okanagan River valley floor. The fan apex has an elevation of 425 m. There have not been any reported significant debris flow events in the past century since the area was settled. There are no obvious surface indications of recent debris flow deposits on the fan, except for small deposits at the fan apex (Jordan et al. 2010).

Testalinden Creek from just below the location of the dam to the fan apex is confined in a steep, V-shaped gully. The overall channel gradient is about 24% (Higman et al. 2011). Testalinden

Figure 4.3 Testalinden Lake (photo credit: Dwayne Tannant)
Creek can be seen in Figure 4.5, captured from Google Earth. Several researchers and engineers conducted field observations after the Testalinden dam failure. Higman and her colleagues recorded their observations by reach from the base of the fan to the top of Testalinden Creek at the location of the failed dam. They described their observations in three parts: upper reach, middle reach, and lower reach (Figure 4.5).

Upper reach: the discharge from the lake flowed in multiple channels across a narrow low-gradient bench before reaching a break in slope, where it became channelized and started to erode the existing channel. The channel sidewalls are generally V-shaped and defined by bedrock. The average channel width is 4 to 5 m, with width increasing downstream. Channel gradient ranges from 35% to short sections reaching 60% below the ridge top (Figure 4.5).

Middle reach: the channel gradients range from 15% to 20%; however, there are some bedrock waterfalls. The channel sidewalls are generally V-shaped. The average channel width is 6 to 7 m.

Lower reach: the creek channel has gradients between 10% and 15% and the substrate is composed of alluvial sediments of unknown depth. Mud on the ravine walls signifies that the debris was between 5 and 7 m deep and moving at roughly 2.6 m/s and 5.1 m/s. The gradient of these reaches was measured by analysing a DEM of the area to compare with Higman’s observations. The upper, middle and lower reaches are shown in Figure 4.5.

The creek gradient is one of the important factors needed to assess debris flow initiation using Equation 20 and 21. The slope of the channel was derived from a DEM of the studied area. The DEM grid cell size is 20 m and they are projected on a NAD 1983 Universal Transverse Mercator zone 11 coordinate system. Two methods were used to determine slope of channel by using ArcGIS software. First, the channel gradient is calculated by spatial and raster analysis. Figure 4.4 presents creek gradient. Second, a longitudinal profile is drawn by using 3D analyst of ArcGIS. The creek profile is given in Figure 4.5.
Figure 4.4 Testalinden creek gradient
Figure 4.5 Testalinden Creek and its longitudinal profile
There are no high resolution DEMs available to extract the cross section of creek before or after the dam failure. The Google Earth DTM also only provides a rough measure of the creek cross-section. The only known parameter is the width of the channel stated by Higman (2011). Unfortunately, there was no description of where the channel width was measured in the cross-section (Figure 4.6). For the thesis, no field recognizance was performed. Therefore, the creek cross section is assumed to have a trapezoid shape.

A survey of the creek after the debris flow by Higman et al. (2011) found that the total quantity of sediment still present in the creek channel after the debris flow had passed through is approximately 80,000 m$^3$. The volume of sediment deposited by the debris flow on the fan was 240,000 to 260,000 m$^3$. Therefore, the volume of sediment in the creek channel before the event was between 320,000 and 340,000 m$^3$.

The length of the creek that contributed sediment to the debris flow is approximately 7 km (Figure 4.5). Assuming the sediment volume in the creek had an approximate trapezoid shape, the average sediment thickness can be calculated. The sediment shape is shown in Figure 4.6. The base of the sediment volume is assumed to be 6 to 10 m wide (and even wider in some locations). The sides of the trapezoid shape are assumed to slope at the angle of repose of the sediments (~37°). Dividing the sediment volume of 320,000 to 340,000 m$^3$ by the channel length of 7000 m yields an average sediment cross-section area of 46 to 48 m$^2$. This value is likely larger in the lower reaches and smaller in the upper reaches. Using these assumptions, the average sediment thickness in the creek is 3.5 m to 4 m.
Figure 4.6 Assumed schematic cross-section of the sediments in the Testalinden Creek before the debris flow

4.2.1 Surficial geology of Testalinden Creek

The surficial geology map of Testalinden Creek presented in Figure 4.7 was created by using shape files for the Okanagan region received from the Ministry of Environment (Finvers 2012). The surficial geology plotted on the map in Figure 4.7 was to determine the sediment type present along each reach of the Testalinden Creek.
Figure 4.7 Surficial geology map of Testalinden Creek

Figure 4.7 shows that a length of 1526 m along the creek is located in moraine, 546 m of the creek has colluvium in the creek bed, and there is no information on the sediment type available for about 4 km along the lower reaches of the creek.

4.3 Climate near Testalinden Dam

4.3.1 Oliver weather station

Environment Canada (2012) archives data from weather stations in Canada in an online format. The Oliver weather station, located 7 km northeast of the Testalinden Creek fan, is the closest active weather station to the Testalinden Dam. The weather conditions recorded during the days leading up to the dam failure are shown in Figure 4.8, Figure 4.9, and Figure 4.10.

Figure 4.8 shows the total precipitation in June 2010 in Oliver. As shown in Figure 4.8, four and five days before the Testalinden event, there was rain in the area. The daily maximum and minimum temperature can be seen in Figure 4.9. Missing data in this figure are for weekends.
when data were not recorded. Based on the daily temperature and precipitation, in June, the area had experienced warming temperatures that caused the snowmelt, and some rainfall. Both of these factors caused the dam to fill and then overtop. In addition, it caused sediment in the creek bed to become saturated and its strength reduced.

Figure 4.8 Daily total precipitation for June 2010 at Oliver Airport weather station
Figure 4.9 Maximum and minimum daily temperature in 2010 at Oliver Airport weather station

Figure 4.10 Monthly total precipitation at the Oliver station in 2010

4.3.2 Mount Kobau Weather station

Archival data for weather measured at the Mount Kobau station located close to Testalinden Dam (Figure 4.5) was gathered and analysed. The last data available from this station were
measured in 1980. In addition, snowpack data were gathered from the Kobau snowpack station that is located very close the Testalinden dam.

Figure 4.11 shows the average monthly minimum, mean, and maximum temperatures and Figure 4.12 shows the average monthly precipitation and snowfall measured at Mount Kobau. Figure 4.12 shows the average snowpack conditions as well as the snowpack data from 2010. Snow pack data are collected from snow pack station that has been working since 1960s close to the dam (Ministry of Environment 2012). A comparison of these figures shows that the snowmelt starts when the monthly mean temperature exceeds 0 °C (near the end of April at an elevation of 1810 m). The peak flows related to snow melt occur about one month later when the monthly minimum temperature reaches approximately 0 °C. Higher precipitation levels in May and June also contribute to the peak inflows to the Testalinden Lake. At high elevations, such as for the monitoring station on Mount Kobau adjacent to Testalinden dam, some snow is still typically present for most of June.

Figure 4.11 Mean, maximum, and minimum monthly temperature at the Mount Kobau weather station (1970 to 1980)
Figure 4.12 Average monthly precipitation and snow fall at the Mount Kobau weather station (1970 to 1980).

Figure 4.13 Mean monthly snow pack at the Mount Kobau snow pack station located at an elevation of 1817 m (1966 to 2012).
The warmer spring weather causing snowmelt combined with a non-functional spillway culvert at the dam is likely the primary cause for the overtopping of the dam.

### 4.4 Testalinden Dam breach

On June 11, 2010, a hiker noticed water flowing across the road that ran along the crest of the dam (Morhart 2010). The report of his observations was not passed along to the appropriate authorities until after the breach occurred at approximately 2:15 pm on June 13, 2010. The breach in the dam released approximately 20,000 m$^3$ of water. The released water descended the creek to the Okanagan valley entraining materials from the creek, initiating a large debris flow, and growing to more than 240,000 m$^3$ in volume by the time it reached the alluvial fan.

The technical reasons for the dam failure have been investigated and it is likely that the combination of late season snowmelt and rainfall, which overtopped the roadway/dam, and an ineffective culvert were contributing factors (Morhart 2010).

Figure 4.14 shows the north side of the breach. This photo was taken on June 14, 2010, one day after the breach occurred.

![Figure 4.14 Breach through Testalinden Dam (photo credit: Dwayne Tannant)](image)

Dr. Tannant surveyed the dam and area around the dam two days after the failure (Figure 4.15).
Figure 4.15 Surveyed breach cross section (Tannant 2010)

A photograph of the breach photo can be seen in Figure 4.16, and a simplified cross section of the breach is given in Figure 4.17. Based on the cross section geometry, the height of water above the bottom of the breach at the time of dam failure was about 2 m and volume of water in reservoir was estimated to be about 20,000 m$^3$. The breach width was more than 6 m as can be seen in Figure 4.15. Because the steep slope bank sloughs after being exposed, the breach width at the time of the dam failure was likely to be smaller than breach width measured two days after the dam failed. So, the effective breach width was assumed to be 6 m.
The peak outflow from the Testalinden dam breach is estimated by using Equation 1 (Froehlich 1995). The height of water at the time of failure was assumed to be 2 m and volume of water in the dam reservoir was 20,000 m$^3$. Equation 1 (Froehlich 1995) predicts a peak breach outflow of 27 m$^3$/s for the Testalinden Dam, which seems to be a reasonable number. Equation 2 predicts a time to reach the peak outflow from the onset of overtopping of approximately 15 minutes. A simplified triangular hydrograph was constructed to meet the constraints of the peak outflow
(Equation 1), time of failure (Equation 2), and reservoir volume, 20,000 m$^3$. This hydrograph is shown in Figure 4.18.

Figure 4.18 Simplified predicted hydrograph for the Testalinden Dam breach

4.5 Analysis of debris flow initiation

Three methods are applied to investigate debris flow initiation in Testalinden Creek: Takahashi (2007), Berti and Simoni (2003), and Yu (2011). These thresholds are explained in Chapter 3.

Takahashi (2007)

According to Takahashi’s work, if the gully is steeper than $\theta_1$ and the unit width discharge is greater than 2 ($q_u \geq 2$), a debris flow will initiate. The following assumptions are made to apply Takahashi’s method.

- The creek is divided into 11 reaches based on the type of sediment in the creek and changes in the creek gradient.
- The channel bedding sediments are assumed to be the same as those around the creek in shown in the surficial geology map (Figure 4.7), except for the upper most reach, which is known to contain bedrock with little sediment.
The frictional strength of the creek channel sediments was extracted from Koloski et al. (1989). Colluvium was assumed to have mean friction angle $\phi = 35^\circ$ and slightly stronger moraine was assumed to have mean friction angle $\phi = 40^\circ$.

The peak outflow of water is assumed constant along the upper reach. The peak in the water flow usually attenuates with travel distance along a creek channel, but attenuation of the hydrograph/peak outflow can be ignored in steep reaches (MFLNRO 2012c). However, entraining of sediment into a debris flow from a channel increases the volume of the flow while a debris flow travels down the creek. Dam flood routing was not performed to address unsteady flow in the channel. Detailed cross-sections of the creek are needed to do this analysis (US Army Corps of Engineers 2010). If more detailed analysis of debris flow generation and evolution are required, then it is recommended that unsteady state flow be modelled along the creek.

In the reaches with no information about surficial geology (Reaches 8 and 11), the sediment type is assumed to be colluvium.
Table 4.1 Testalinden Creek debris flow initiation based on Equation 5 and 6

<table>
<thead>
<tr>
<th>Channel reach</th>
<th>Length (m)</th>
<th>Channel sediment</th>
<th>Channel slope (°)</th>
<th>$q_0$ (m$^3$/m.s)</th>
<th>$q^*$</th>
<th>$\theta_1^*$</th>
<th>Debris flow initiation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>126</td>
<td>Bedrock</td>
<td>9</td>
<td>5.3</td>
<td>152.0</td>
<td>6</td>
<td>No</td>
</tr>
<tr>
<td>2</td>
<td>1651</td>
<td>Colluvium</td>
<td>19</td>
<td>5.3</td>
<td>75.1</td>
<td>5</td>
<td>Yes</td>
</tr>
<tr>
<td>3</td>
<td>35</td>
<td>Moraine</td>
<td>13</td>
<td>5.3</td>
<td>46.6</td>
<td>6</td>
<td>NA</td>
</tr>
<tr>
<td>4</td>
<td>16</td>
<td>Colluvium</td>
<td>20</td>
<td>5.3</td>
<td>32.4</td>
<td>5</td>
<td>NA</td>
</tr>
<tr>
<td>5</td>
<td>91</td>
<td>Moraine</td>
<td>21</td>
<td>5.3</td>
<td>24.3</td>
<td>6</td>
<td>NA</td>
</tr>
<tr>
<td>6</td>
<td>224</td>
<td>Colluvium</td>
<td>7</td>
<td>5.3</td>
<td>19.0</td>
<td>5</td>
<td>NA</td>
</tr>
<tr>
<td>7</td>
<td>95</td>
<td>Moraine</td>
<td>10</td>
<td>4.4</td>
<td>12.8</td>
<td>6</td>
<td>NA</td>
</tr>
<tr>
<td>8</td>
<td>196</td>
<td>No data</td>
<td>11</td>
<td>4.4</td>
<td>10.7</td>
<td>5</td>
<td>NA</td>
</tr>
<tr>
<td>9</td>
<td>1179</td>
<td>Moraine</td>
<td>10</td>
<td>4.4</td>
<td>9.1</td>
<td>6</td>
<td>NA</td>
</tr>
<tr>
<td>10</td>
<td>1213</td>
<td>Colluvium</td>
<td>9</td>
<td>4.4</td>
<td>5.6</td>
<td>5</td>
<td>NA</td>
</tr>
<tr>
<td>11</td>
<td>4143</td>
<td>No data</td>
<td>6</td>
<td>4.4</td>
<td>1.7</td>
<td>5</td>
<td>NA</td>
</tr>
</tbody>
</table>

Based on the assumptions and Takahashi’s threshold equations, the result of the analysis given in Table 4.1 shows that $q_i$ is greater than the critical value of 2 for every reach except Reach 11. Reach 1 does not have any significant sediment over the bedrock in the channel, so a debris flow cannot. Reach 2 is the first location below the dam where a debris is predicted to initiate (sediment is present, channel slope is much greater than $\theta_1$, and $q_i$ is much greater than 2). The table suggests that debris flow initiation occurred in Reach 2 and continue to grow as it travelled down to Reach 11. However, in term of debris flow initiation, predictions made downstream from the first reach in which a debris flow is triggered are not applicable.

**Berti and Simoni (2003)**

Berti and Simoni (2003) stated that when the Factor of Safety, $FS$, is less than unity, a debris flow can initiate. The assumptions made to apply this method are similar to Takahashi’s method.
In addition, $\beta$, which is the direction of the groundwater flow vector, is assumed equal to the channel slope. The sediment lining the creek channel is assumed to have a uniform thickness across the channel. Based on Section 3.2.2, the ratio of the depth of sediment to the height of water in channel should be at least 1 for the mobilized sediment to be classified as a debris flow. The sediment is also assumed to be saturated with zero cohesion. To calculate the maximum height of water flowing down the creek, the dam outburst flow was estimated from Equation 1. Manning's roughness coefficient $n$ is taken as 0.0315 for a stream channel with gravel bedding (Arcement and Schneider 1989). As shown in Table 4.2, the predicted outflow of 30 m$^3$/s is expected to create a peak height of water flowing down the creek channel of 0.8 to 0.6 m in the first two reaches below the dam.

The values of $FS$ less than unity in Table 4.2 indicate reaches where a debris flow is predicted to initiate. The $FS$ is lower than 1 in Reach 1 and Reach 2. Because Reach 1 had little sediment, a debris flow can only occur in Reach 2 and beyond. After a debris flow initiates, the growing volume of entrained material changes the flow characteristics and hence the methodology is no longer applicable. Figure 4.19 shows the location where the debris flow is predicted to have been triggered.
Table 4.2 Testalinden Creek debris flow initiation based on Equation 7

<table>
<thead>
<tr>
<th>Channel reach</th>
<th>Length (m)</th>
<th>Channel slope (%)</th>
<th>Channel sediment</th>
<th>Sediment depth (m)</th>
<th>Water height (m)</th>
<th>FS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>126</td>
<td>16</td>
<td>Bedrock</td>
<td>≈ 0</td>
<td>0.8</td>
<td>No</td>
</tr>
<tr>
<td>2</td>
<td>1651</td>
<td>34</td>
<td>Colluvium</td>
<td>0.5 to 2</td>
<td>0.6</td>
<td>0.7</td>
</tr>
<tr>
<td>3</td>
<td>35</td>
<td>22</td>
<td>Moraine</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>4</td>
<td>16</td>
<td>36</td>
<td>Colluvium</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>5</td>
<td>91</td>
<td>36</td>
<td>Moraine</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>6</td>
<td>224</td>
<td>13</td>
<td>Colluvium</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>7</td>
<td>95</td>
<td>18</td>
<td>Moraine</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>8</td>
<td>196</td>
<td>20</td>
<td>No data</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>9</td>
<td>1179</td>
<td>17</td>
<td>Moraine</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>10</td>
<td>1213</td>
<td>16</td>
<td>Colluvium</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>11</td>
<td>4143</td>
<td>10</td>
<td>No data</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>
Figure 4.19 Debris flow initiation predicted to occur in the upper reach of Testalinden Creek
Yu (2011) reported a method to determine the diameter of coarse particles mobilized by discharge in a channel as explained in Chapter 3. The channel discharge varies depending on the assumed channel width. It should be mentioned that lag and attenuation of peak outflow are not considered in this study. Equation 8 is applied for the Testalinden breach discharge and results are given in Table 4.3. Equation 8 predicts that sediment particles as large as 120 mm can be carried by the outflow from the dam breach in the upper section of the Testalinden Creek. Therefore, it is reasonable to expect that debris flow initiation can occur for the peak discharge velocities in the channel.

Lower down the creek and on the fan in the valley bottom, boulders much larger than this size were carried along the creek on June 13, 2010 (Figure 4.20), but these boulders were transported by a much larger debris flow than the dam breach water outflow used here to estimate initiation of a debris flow.

Table 4.3 Particle diameter mobilized by channel discharge

<table>
<thead>
<tr>
<th>$q_0$ (m$^3$/m s)</th>
<th>$d_0$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.8</td>
<td>121</td>
</tr>
<tr>
<td>4.8</td>
<td>107</td>
</tr>
</tbody>
</table>
4.5.1 Discussion of Testalinden debris flow initiation results

The three debris flow initiation threshold methods all suggest that the discharge from the Testalinden Dam was capable of triggering a debris flow in Reach 2. Since most of the creek is confined to a V-shaped valley that has a slope greater than 10°, a debris flow was easily able to initiate and grow as it travelled down toward the valley bottom.

Figure 4.21 shows Google Earth aerial photographs captured before and after the dam failure. By comparing images (a) and (b), evidence of the debris flow is visible. The highlighted circle indicates a point along the channel where evidence of debris flow scour of the channel becomes more obvious. This location is near the confluence of a tributary along the Testalinden Creek and where it is expected that more substantial quantities of sediment and debris were present in the channel. Starting at this point, evidence of scour and vegetation removal starts and continues down the channel all the way to the fan. This spot is located at the lower end of Reach 2, 1.6 km downstream from the dam.

Figure 4.22 shows two closer views of the creek channel. A scar along the creek where vegetation has been removed by the debris flow can been seen within the red outlined areas. In addition, exposed banks of glacial till are clearly evident. These are one source of sediment that
lined the creek channel before the debris flow occurred. The initial stages of debris flow initiation and growth probably occurred slightly upstream of this location. There is a transition from minor evidence of tree removal to a continuous scar as the creek travels from left to right across the images.

Figure 4.21 Probable debris flow initiation location
4.6 Sensitivity analysis

To investigate the effect of uncertainty of input data in the Berti and Simoni method (Equation 7), a sensitivity analysis is done for Reach 2 of Testalinden Creek. A sensitivity analysis is the process of estimating how a calculated parameter changes as values of input parameters are changed. Sensitivity analysis can identify the input parameters that have the most influence on model output. Sensitivity analysis can also quantify the change in output caused by uncertainty and variability in the values of input parameters. To perform the sensitivity analysis, Oracle Crystal Ball Software was used. The slope of the reach, height of water, depth of sediment, and friction angle are parameters that were assessed. The distributions of these input parameters were determined or estimated.

The gradients of short sections along Reach 2 in Testalinden Creek were extracted from the Geo Base DEM. The DEM cell size is 20 m. The slope (gradient) was determined in 5-cell (~100 m)
intervals to determine the distribution of gradient along the creek. A histogram of the creek gradients is presented in Figure 4.23. Based on this figure, the distribution of gradient can be approximated by a normal distribution with a mean of 20° and a standard deviation equal to 14°.

![Histogram of Creek Gradients](image)

**Figure 4.23 Slope distribution for Reach 2**

The coefficient of variance for the friction angle for most soil types has been found to be approximately 10% (Phoon and Kulhawy 1999). The friction angle is typically normally distributed and the distribution of the friction angle for the creek bed sediments was assumed to range from 30° to 40°. Using a 10% coefficient of variance, the standard deviation for the normal distribution was determined.

The height of water in the creek is assumed to be uniformly distributed from 0 m to 1.5 m. There is no information available about the distribution of sediment depth in the creek channel. The sediment depth is also assumed to be uniformly distributed over a range 0 to 5 m.

The calculated $FS$ is most sensitive to the creek gradient as it contributes more than 50% to the overall variance. The depth of the creek sediments contributes about 20 to 25% to the variance in $FS$. The height of water in the creek has low impact on the $FS$ variance (approximately 10%). The $FS$ is least sensitive to the friction angle of sediments (<10%).
Chapter 5 Glanzier Case Study

The other case study that is investigated in this research is the Glanzier Dam. This dam is located high above the valley near Armstrong, BC. A recent survey by a senior dam safety officer identified a potential debris flow hazard if the dam were to fail. Therefore, the developed methodology is applied to check his hypothesis.

5.1 Glanzier Creek watershed and Glanzier Dam

The Glanzier Creek watershed covers an area of 11 km$^2$ and extends in elevation from 360 m at the confluence with Fortune Creek to approximately 1500 m at the eastern boundary (Figure 5.1). The watershed is in the Shuswap Highlands Physiographic unit (Dobson Engineering 1997).

Five sources of data were used to analyse the debris flow hazard associated with a potential failure of Glanzier dam:

- observations and photographs of the dam and creek obtained from the dam safety officer
- Google Earth aerial photographs
- photographs of the reservoir posted on Google Earth
- copy of the water licence for the dam (Department of Lands, Forests, and Water Resources 1964)
- surficial geology ‘shape file’ maps (Finvers 2012).

Glanzier Dam was built before the early 1960's (Department of Lands, Forests, and Water Resources 1964). It is an earth fill dam located at an elevation of 1465 m. The dam height is 3 m and the licensed dam reservoir capacity is 24,670 m$^3$ (Noseworthy 2012). Figure 5.2 shows the photograph taken in 2012.
Figure 5.1 Glanzier Creek (modified from Atlas of Canada 2012)
5.2 Creek gradient and cross-section

Glanzier Creek travels 6.5 km from the dam to Highway 97A and the average creek gradient is 17%. The gradient of the creek downstream of the dam was obtained by digitizing the Google Earth DTM. As seen in Figure 5.3, in the first 500 m downstream from the dam, the creek has low gradient (5%); after that, the creek is quite steep (average 25%) until it reaches to the alluvial fan.

The Glanzier Creek profile has similarities to Testalinden Creek with a short low gradient reach at the very top of the watershed that makes a sudden transition to a steep gradient as the creek flows over the edge of the upland plateau and enters the main valley below. In addition, the dams on both creeks are located near the top of the watershed on a short low gradient reach. Furthermore, the dams are located only a short distance upstream of the abrupt transition from low to high gradient sections of the creek.
The creek profile was divided to 11 reaches based on the type of sediment obtained from Figure 5.4 and channel gradient. The reaches are shown in Figure 5.3.

Very little information was available about the creek channel width. The Google Earth aerial photo indicates a narrow valley downstream of the dam. It is assumed that the channel width is 5 m in the upper reaches of the creek and 6 m in the lower reaches, which is again similar to Testalinden Creek.

### 5.1 Surficial geology and creek bed sediments

A surficial geology map of Glanzier Creek is presented in Figure 5.4. This map was made using data received from the BC Ministry of Environment (Finvers 2012) that were clipped and projected onto the Glanzier Creek area. The map was used to determine the sediment type in the creek bed. Less than 2 km of the creek is in moraine and about 4 km of creek is embedded in colluvium.

In Reach 8 and 11 there is no information about the surficial geology; here the material type is assumed to be colluvium. The friction angle assumed applicable for the moraine \((\phi = 40^\circ)\) and colluvium \((\phi = 35^\circ)\) were extracted from Koloski et al. (1989). In the absence of any information, the depth of the sediment in the channel was assumed 1 m for the upper reaches.
Figure 5.3 Glanzier Creek and its longitudinal profile
The first step in analysing the potential for debris flow initiation caused by a possible dam failure is to estimate the peak outflow and breach hydrograph for failure of the Glanzier dam. A number of assumptions are made to estimate the breach outflow hydrograph. Overtopping in the spring is assumed to be the cause of the dam failure. The height of water at the time of failure is equal to the dam height, 3 m. The lake volume at the time of failure will be larger than the licenced water supply capacity (24,670 m$^3$). The dam likely required a freeboard of approximately 1 m above full supply level (U.S. Bureau of Reclamation 1981). The lake area is roughly 12,430 m$^2$. Assuming the dam fails by overtopping, the volume of water in the reservoir will be 12,430 m$^3$ higher than the licenced water supply capacity. A value 37,000 m$^3$ is assumed for the volume of the reservoir $V_w$ at failure. All the water in the lake is assumed to flow out through the dam breach, and the breach shape is assumed to be rectangular.

As discussed earlier when comparing various empirical approaches for predicting peak breach outflows and failure times for the Testalinden dam, Equations 1 and 2 are assumed to give the
best estimate of the peak outflow and time of failure for small earth dams. Equation 1, gives a peak discharge of approximately $52 \text{ m}^3/\text{s}$. Equation 2 predicts a time to reach the peak outflow from the onset of overtopping of approximately 15 minutes.

Using the reservoir volume at the time of overtopping (37,000 m$^3$), the time to drain the lake and the overall shape of the breach hydrograph can be estimated by ensuring the area under the hydrograph is equal the reservoir volume at the point of failure. Assuming a simple triangular shape, the hydrograph shown in Figure 5.5 is generated.

![Figure 5.5 Estimated hydrograph for a breach of Glanzier Dam](image)

**5.3 Analysis of debris flow initiation**

To analyse the potential for debris flow initiation, the methodology outlined in Figure 3.5 was used. The predicted peak breach outflow ($52 \text{ m}^3/\text{s}$) and the assumed for a trapezoid channel cross section were used to estimate the height of water flowing in the channel, $h_0$, via Manning's equation (Equation 9) for stream flow. The Manning's coefficient, $n$ is taken as 0.0315, which is appropriate for stream channels containing gravel. The direction of the flow vector in the sediment, $\beta$, is assumed equal to the channel slope.
Equation 7 was used to determine the factor of safety for debris flow initiation. The results are shown in Table 5.1. Based on this table, if a dam breach occurs the outflow will not trigger a debris flow in the first two reaches, which have a low channel gradient. However, once the outflow travels 540 m from the dam and encounters the steeper reaches of the creek, debris flow initiation is predicted to occur. After this reach, due changes in the flow from water to debris flow, this method is not applicable for the rest of the reaches.

The outflow from a dam breach would generate a specific discharge, \( q_0 \) of 8.6 to 10.4 m\(^3\)/m\( \cdot \)s depending on the assumed width of the channel (6 or 5 m). Using this discharge and the Equation 13 from Yu (2011), the predicted breach outflow could mobilize sediment particles with a size of up to roughly 167 to 190 mm. Judging from the photograph taken of an old bouldery deposit in the Glanzier Creek channel shown in Figure 5.6, the creek may have experienced debris flows carrying boulder-sized particles in the past. The release of water from a dam breach would be equivalent to a very extreme meteorological event on this creek and hence creation of a destructive debris flow should be expected.
Table 5.1 Glanzier Creek reach properties and predicted factor of safety for debris flow initiation

<table>
<thead>
<tr>
<th>Channel reach</th>
<th>Length (m)</th>
<th>Channel slope (%)</th>
<th>Channel bed geology</th>
<th>Height of water (m)</th>
<th>FS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>110</td>
<td>5</td>
<td>Moraine</td>
<td>2.2</td>
<td>2.4</td>
</tr>
<tr>
<td>2</td>
<td>432</td>
<td>7</td>
<td>Colluvium</td>
<td>2.0</td>
<td>1.4</td>
</tr>
<tr>
<td><strong>3</strong></td>
<td><strong>958</strong></td>
<td><strong>37</strong></td>
<td>Moraine</td>
<td><strong>1.1</strong></td>
<td><strong>0.2</strong></td>
</tr>
<tr>
<td>4</td>
<td>500</td>
<td>26</td>
<td>Colluvium</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>5</td>
<td>500</td>
<td>23</td>
<td>Moraine</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>6</td>
<td>500</td>
<td>14</td>
<td>Colluvium</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>7</td>
<td>500</td>
<td>18</td>
<td>Moraine</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>8</td>
<td>500</td>
<td>13</td>
<td>No data</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>9</td>
<td>500</td>
<td>17</td>
<td>Moraine</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>10</td>
<td>500</td>
<td>20</td>
<td>Colluvium</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>11</td>
<td>250</td>
<td>10</td>
<td>No data</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>
Figure 5.6 Boulders in the Glanzier Creek channel downstream of the dam
(photo credit: Ministry of Forests, Lands and Natural Resource Operations, BC)
Chapter 6 Conclusions

This study highlights a situation where a small earth dam on a steep creek perched high above a main valley creates a significant debris flow hazard. If the dams fail suddenly releases stored water, which then triggers a debris flow, the resulting debris flow can be far larger and more destructive than a flood of water. Flood hazard assessment is what has been conducted to date for dams when assessing the consequence of dam failure as recommended by the Canadian Dam Association. This thesis demonstrates a class of problems where debris flow initiation is more important and must be considered when determining the failure consequence of a dam.

Due to the large number of privately owned small earth dams in British Columbia and a lack of rigorous monitoring of these dams, debris flow initiation triggered by dam breaching should be included in future hazard assessment assessments and downstream analysis.

The dam-break and debris-flow-initiation methodology developed in this thesis (Section 3.4.5) provides a means to assess debris flow initiation with minimal data requirements. This study used readily available digital elevation models, aerial photos, geological maps, and published reports and literature. The key input parameters needed to assess whether a debris flow can initiate below a dam breach are:

- dam breach peak outflow or the breach hydrograph,
- creek gradient and creek cross-section geometry, and
- properties of the creek bed sediments (shear strength, grain size, sediment depth).

Froehlich’s relations (Equation 1 and 2) are used to predict dam breach peak outflow and the time of failure using the height of the dam and reservoir volume as inputs for these relations. Dam height and reservoir volume are extracted from an online database provided in Google Earth format (MFLNRO 2012b).

The creek gradient and cross-section shape are determined from spatial analysis of a digital elevation model. To determine the nature of the sediments in the creek channel, surficial geology maps, field investigation reports, and Google Earth aerial photos are used.

The methodology presented in this thesis could be implemented in GIS-based software for spatial and raster analysis of debris flow hazards posed by dam failure. The developed
methodology has the potential to assist Dam Safety Officers in BC in rapidly assessing debris flow hazards posed by potential dam failures.

Finally, the following suggestions are recommended to enhance this methodology.

- Collect all dam failures records in British Columbia to validate or refine the empirical equations for predicting peak outflow and the breach formation time. Record the geometry and materials used to construct the dams to increase the certainty of input data used to predict the hydrograph for a potential breach.
- Conduct field investigations to record the depth and type of debris in channels where dams with high risk of failure are located. Also, measure the channel cross-sections to improve the certainty of this method.
- It would be useful to develop numerical-spatial model for implementation in ArcGIS to allow rapid assessment of all dams in a specific watershed (e.g. Okanagan watershed) to create a debris flow hazard map.
- It would be helpful to extend the analysis beyond debris flow initiation and include aspects of debris flow growth (entraining materials in a flow) and deposition along the creek channel, and to consider debris flow growth and dam flood routing together.
References


Finvers, M. 2012. Personal communication.


Noseworthy, M. 2012. Personal communication.


