University of Alberta

Quantitative Risk Assessment of Natural and Cut Slopes: Measuring Uncertainty in the Estimated Risks and Proposed Framework for Developing Risk Evaluation Criteria

by

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ABSTRACT

Understanding and limiting the risks inherent to natural and cut slopes are now recognized to be a priority in achieving an acceptable quality of life. Various methods of risk management that have been proposed in the last three decades have evolved into a general framework for landslide risk management. In particular, quantitative risk assessments can assist in communicating risks. They also provide a clear and systematic framework to analyze slope failure processes, from origin, to movement, to consequence; and the effect of different remedial works and strategies.

Some of the challenges and perceived limitations of quantitative risk assessments are related to the necessary input of expert opinion when estimating the risk levels in a quantitative manner. One objective of this work is the systematic assessment of the uncertainties in the estimated values of risk. Quantitative risk analyses are carried out for two case histories, where population of the analyses input parameters is done as probability distributions rather than fixed values. The probability distributions of the input parameters cover the range of values believed realistic for each input parameter. The risk is then estimated through a Monte Carlo simulation technique, and the outcome of the analysis is a probability distribution of the estimated risk. This methodology shows the potential for evaluating the uncertainties related to risk estimations.

The full potential of the risk management framework is best met with the establishment of risk evaluation criteria. The other objective of this work focuses on the development of risk evaluation criteria. It is not the intention of this work to develop case specific criteria, as this responsibility should lie with owners and regulators, but to propose a framework for developing the criteria, where the risk analyst takes an active role.

A summary of the state of practice for quantitative risk assessments is included as part of the thesis. The work on the evaluation of uncertainty related to the estimated risks and a proposed framework for developing risk evaluation criteria are then presented. The last two chapters of the thesis present a summary of the research results, conclusions and proposed future research.

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Samanta, my dear, this is for you, for you have illuminated my life since the day you were born. You save me every time just by thinking of your laughter.

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CHAPTER 1: INTRODUCTION

Geotechnical engineering is fundamentally about managing risk (Ho *et al*. 2000). Understanding and limiting the risks inherent to natural and cut slopes is now recognized to be priority in achieving an acceptable quality of life. It might seem we are now more sensible about the existence of these risks than in the past, but as noted by Fell and Hartford (1997), studies of natural and man-made slopes have always involved some form of risk assessment.

Historically, management of risks associated with natural and cut slopes has been done through the adoption of a variety of approaches. In most early cases, risk assessments were only implicit and subjective within the decision making process for slope management. Today, several approaches are available for the practitioner. These range from relying on past performance of similar structures (which evolved into empirical design guidelines), to estimations of safety factors, probability of failure, and slope deformation monitoring together with the adoption of displacement thresholds and early warning systems.

Comprehensive methods of risk management incorporating measures of failure consequences have been proposed in the last three decades. These have evolved into a general framework for landslide risk management, where the trend is to estimate the risks, evaluate the risks and manage/mitigate the risks. The framework has also been conceived as an iterative and continuous process. In this regard, Morgenstern (1995) noted that risk assessments don't need to be quantitative, and management of these risks can successfully proceed without their quantification. However, quantitative risk assessments (QRA) can assist in communicating risks and the full potential of the framework is best met with the establishment of risk evaluation criteria. The QRA process also provides a clear and systematic framework to analyze the entire slope failure process, from origin, to movement, to consequence; and the effect of different remedial measures and strategies (Einstein 1997).

There is still a lack of general acceptance of the method by the profession (Fell *et al*. 2005). The benefits and the limitations of QRA applied to slopes are discussed in IUGS (1997), Fell *et al*. (2005) and Ho *et al*. (2000). It is clear that the limitations of the framework need to be addressed systematically in order to achieve its full potential and acceptance. The present research addresses some of these limitations, in a comprehensive but simple manner, that can be readily applied by the practitioner.

Moreover, publication of case histories emphasising the application of the method has been requested as a means to encourage its use among practitioners (Whitman 1984, Morgenstern 1995, Ho *et al*. 2000). The present research study was developed through worked case histories, as a response to such requests.

1.1. BACKGROUND

Natural and engineered slopes pose potential hazards to the public, workers, infrastructure, economy and the environment. The Frank Slide, southwestern Alberta, occurred in 1903, buried half the town of Frank and killed more than 70 people (Cruden and Martin 2007). The Hope slide, in southern British Columbia, occurred in 1965 and buried 4.5 km of Highway 3 (Bruce and Cruden 1977). Four people were killed as a result of the slide. Hong Kong confronts an acute landslide problem in natural and engineered slopes in relation to urban development (Wong and Ho 2007), and so does Malaysia (Othman *et al*. 2007), China (Yin *et al*. 2007), and Norway (Lacasse and Nadim 2007).

Transportation corridors through mountainous terrain are also highly exposed to natural and cut slope instabilities (Brawner 1978, Pierson 1992, Bunce *et al*. 1997, Budetta 2004, Lan *et al*. 2010, Hungr and Evans 1989, Evans and Hungr 1993). A well known case in Canada occurred in 1982, when a rock block fell on a vehicle killing a woman and disabling her father while they were delayed in traffic on British Columbia's Highway 99 (Bunce *et al*. 1997).

There is a long history of instabilities in natural slopes and cuts along the Canadian Pacific Railway (CP) and the Canadian National Railway (CN). Mackay (1997) presented an overview of CP's intense rock slope management program implemented following a fatal derailment in British Columbia in 1974.

These few examples illustrate the risks posed to society in relation to its interaction with natural and engineered slopes. It is clear that these risks need to be addressed in an effective and efficient manner given the limited resources generally available for risk mitigation. An important constraint is that quantitative prediction of behaviour in such problems, even under ideal circumstances, may not be reliable (Morgenstern 2000), and deterministic analyses have shown great variability with respect to actual behaviour.

Terzaghi, for one version of the introduction for his book Soil Mechanics in Engineering Practice stated that vast efforts go into securing only roughly approximate values for the physical constants that appear in the equations, and that many variables remain unknown (Peck 1969). He further stated that the results of computations are not more than working hypotheses subject to confirmation during construction. These uncertainties were dealt with by adopting excessive factors of safety, in a wasteful manner, or making assumptions in accordance with the general experience, which can be dangerous. In this introduction to his Observational Method, Terzaghi was stating the issues related to uncertainty and risk in geotechnical engineering.

Casagrande in the 2nd Terzaghi Lecture discussed the role of risk in earth work and foundation engineering (Casagrande 1965). He considered that calculated risk consisted of two steps. First, the use of imperfect knowledge, guided by judgement, to estimate probable ranges of the variables that enter into the solution of a particular problem. Second, deciding on appropriate margins of safety, or degree of risk, considering economic factors and the magnitude of losses as a result of failure.

Whitman's Terzaghi Lecture synthesized the state of the art of the application of probability theory and risk analysis to geotechnical problems (Whitman 1984). This lecture is considered the next major milestone in attempts to quantify risk (Morgenstern 1995).

Morgenstern (1995) summarized risk assessment concepts using the framework for risk management adopted by the Canadian Standards Association (CSA,1991), presented here in Figure 1-1.

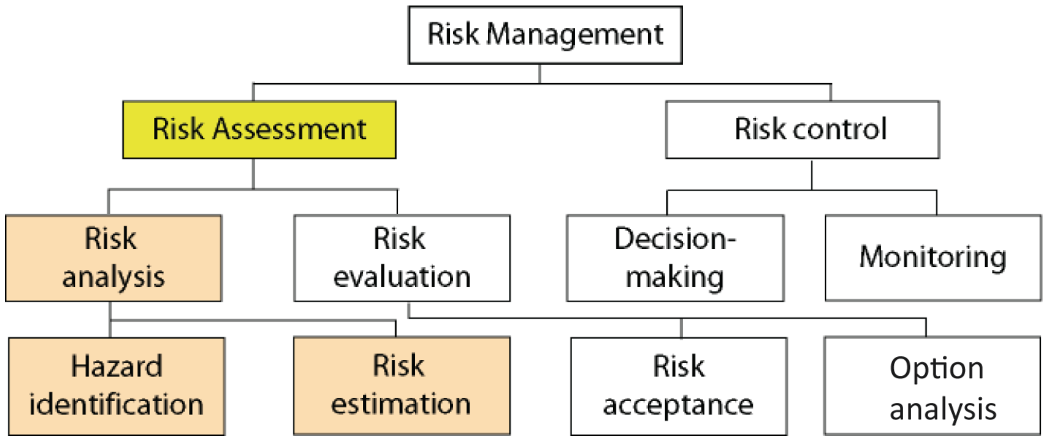


Figure 1-1 A framework for risk management (After CSA, 1991)

In this framework the risk assessment consists in the identification of the hazards, estimation of the associated risks (risk analysis) and evaluation of those risks against adopted criteria.

The Landslide Risk Management Conference held in Vancouver in 2005 presents a series of QRA state of the art papers (Hungr *et al*. 2005). These include a synthesis of the QRA framework as well as the tools available to develop the analyses. This QRA state of the art is the topic of Chapter 2 of this thesis.

1.1.1. Use of Qualitative Or Quantitative Risk Analyses

Qualitative risk assessments are carried out using ranking methods that vary in detail and complexity. They usually employ instability scores representing the relative probabilities of failure and consequence scores representing the relative severity of the consequences of failure (Morgenstern 1997).

Qualitative risk assessments of this kind satisfies many needs in practice, particularly where relative rankings for zonation or resource allocation are needed (Morgenstern 1997). Discussion of qualitative risk assessments and tools available for their application to slope instability problems are presented by Lee and Jones (2004). Some of these common tools used for qualitative risk analyses are What if/then analyses, Failure Modes and Effects Analysis (FMEA), Risk Registers, and Risk Matrices.

Lee and Jones (2004) also discussed some of the limitations of qualitative analyses:

- The use of subjective scales to rank hazards and consequences can be problematic. Perceptions of what actually constitutes high or low risk can vary considerably, leading to miscommunication between professionals and other individuals involved in the study,
- Difficulties establishing whether the identified risk levels are acceptable and the real urgency for remedial measures to be in place. This can result in legal consequences to the specialist if the hazard is realized, as discussed by Leroi *et al*. (2005), and;
- Difficulties establishing overall risk levels at sites where there are multiple hazards, each with different frequency and potential consequences.

These limitations become significant when qualitative analyses are taken beyond the limits of their capabilities. This is a consequence of the industry and society requiring increased knowledge and information for decision-making regarding public safety and financial investment. Morgenstern (1997) noted a growing pressure on the geotechnical engineer to apply QRA to landslide problems. Some of the sources he identified of this growing pressure were:

- Agencies outside of the geotechnical community are developing risk criteria that influence the work of the professional engineer and his professional liability. These agencies are requiring more and more statistical responses rather than safety factors,
- Some agencies are adopting probabilistic methods as a basis for their planning studies, as is the case of the U.S. Corps of Engineers, which uses risk-based decision tools for planning studies related to rehabilitation of water resource projects,
- Courts are disposed to express tolerable risk. This is illustrated by Mr. Justice Thomas Berger's 1973 decision that for a potentially catastrophic landslide affecting a proposed subdivision, a return period of 10 000 years was not acceptable (Porter and Morgenstern 2012).
- Loading conditions and material parameters are frequently stated in probabilistic terms. This drives to statistical answers regarding structure safety (failure probability and design reliability), and;
- Clients welcome quantitative risk analyses as a means of understanding their exposure to landslide hazards and establishing priorities with regard to mitigation.

As noted by Morgenstern (1995), risk assessments don't need to be quantitative for risk management to proceed successfully. The choice between qualitative and quantitative analyses will depend on the available data, the experience of the specialist and the purpose of the analysis (Leroi *et al*. 2005). However, it is clear that the limitations of qualitative analyses and the increasing tendency of society to communicate and make decisions based on quantified risks, drives to the adoption of quantitative analyses for an increasing number of cases.

A generic QRA framework for natural and engineered slopes now exists. This is being adopted by the geotechnical community that has accepted QRA as a main tool for slope risk management. This framework and details on the methods that can be used for quantification of each step are provided by Fell and Hartford (1997), IUGS (1997), Einstein (1997), ERM (1998), Ho *et al*. (2000), Crozier and Glade (2005), Lee and Jones (2004), Leroi *et al*. (2005), Picarelli *et al*. (2005), AGS (2007). Chapter 2 of this thesis presents a review of the QRA state of practice.

1.1.2. On the Challenges and Perceived Limitations of QRA

Ho el at. (2000) and Fell *et al*. (2005) present a discussion of the perceived limitations of QRA, based on IUGS (1997). These can be summarized as follows:

- The necessary input of judgement into the analysis may result in considerable uncertainty inherent to the estimated risks,
- Revisiting an assessment can lead to significant change in light of increased knowledge or the adoption of different approaches,
- Difficulties recognizing all potential hazards, thus, underestimating the risk,
- Results of an assessment are seldom verifiable,
- Issues regarding the adoption of acceptable and/or tolerable risk criteria,
- The variety of approaches that can be adopted to estimate risks (landslide frequencies and consequence probabilities) can result in substantially different estimations by different practitioners,
- QRA costs may outweigh the benefit of the method for decision making purposes, and;
- The difficulties in estimating risks with low occurrence probability.

In their discussion, Ho *et al*. (2000) noted that most of the perceived limitations are also valid for other methods (the use of factors of safety and empirical guidelines). It is recognized, however, that the extent of these may have greater impact on QRA. Regarding costs, the objective is to make use of the tool best suited for the assessment. QRA might not be the method of choice for routine problems, however more complicated situations could benefit from its application.

1.2. RESEARCH OBJECTIVES, METHODOLOGY AND ORGANIZATION

The research focuses on QRA for natural and cut slope hazards in relation to the challenges and perceived limitations discussed above. The main objectives of the research can be summarized as follows:

- Present detailed QRA case studies. Each step of the analyses should be clearly defined. The thinking behind building and populating the process model defined to estimate the risk values should also be clear. The case studies should be comprehensive in the treatment of all factors affecting the estimated risks (within the scope defined), but simple enough to be readily applicable by the practitioner,
- Assess the influence of the uncertainties related to the necessary input of subjective probabilities in QRA. The objective is to demonstrate the increased value of QRA when better understanding the level of uncertainty in the result. It is stressed here that this uncertainty exists regardless the method of analysis chosen, and that QRA presents a means of assessing it, even if only partially or in a semi-quantitative manner,
- Illustrate the method's potential to highlight the weak areas of knowledge related to the variables affecting the estimated risks, both within the hazard and consequence analyses. This can help define future studies that would more efficiently reduce the levels of uncertainty,
- Illustrate the method's potential as a tool for resource allocation and decision making regarding the adoption of risk mitigation strategies. QRA analysis of the processes leading to a loss would highlight the step where mitigation measures result in larger reductions in risk. Moreover, it highlights the riskier scenarios in a quantitative manner, and;
- Propose a simple but comprehensive framework for the development of risk evaluation criteria. It is believed the risk analyst, having the most knowledge on the temporal and spatial characteristics of the hazard and elements at risk, should be actively involved in the development of the risk evaluation criteria. Final decision, and liability, for the risk levels to be considered acceptable or tolerable should be the responsibility of the local government, regulator agency and/or operator. As such, the risk levels of the case histories developed here are compared against previously proposed criteria. It is not the purpose of this research to propose case-specific risk evaluation criteria for the case studies analysed.

The proposed methodology consists in developing the research study through worked case histories. Each case will include:

- Hazard analysis. This includes description of the extent of the instability, failure mechanisms, movement triggers, occurrence probability and post failure analyses,
- Consequence analysis. This consists in defining and characterizing the elements exposed, estimating the spatial and temporal probabilities that the exposed elements will be hit given the hazard is realized, and estimating the value of the potential loss (this last one will not be included for risk to life estimations),

- Measure of the estimated risk uncertainty related to the input of subjective probabilities, and;
- Evaluation of the estimated risks against adopted/proposed criteria.

Chapter 2 summarizes the current QRA state of practice and presents some definitions necessary to better understand the chapters to follow. Chapter 3 and Chapter 4 present the two case studies developed. A separate chapter (Chapter 5) is devoted to discuss the development of risk evaluation criteria. A summary of this research and its findings is presented in Chapter 6. Conclusions and future research are presented in Chapter 7.

The main focus of the case studies presented in Chapters 3 and 4 is the systematic assessment of the uncertainties in the estimated values of risk, given lack of information / knowledge, which require the input of subjective probabilities. As such, the main discussions are directed towards the proposed methodology to assess and manage these uncertainties in risk estimations.

CHAPTER 2: QUANTITATIVE RISK ASSESSMENT OF NATURAL AND CUT SLOPES - STATE OF PRACTICE

Management of risks associated with natural and cut slopes requires understanding the levels of risk posed by these hazards and assessing their tolerance. It was previously noted that these assessments could be undertaken following a qualitative or quantitative approach. The advantages of the quantitative approach (Quantitative Risk Assessment - or QRA), were outlined in Chapter 1. This section presents an overview of the current state of practice for QRA in the context of natural and cut slopes. Some of the concepts necessary to better understand the chapters that follow are also included at the end of the chapter.

Figure 2-1 presents the current risk management framework after Fell *et al.* (2005). Minor modifications from the original are made for illustrative purposes only. The focus of this Chapter is the risk assessment component, which includes analysing the risks and evaluating them against some adopted criteria. A brief comment of the requirements to take QRA into risk management is also presented.

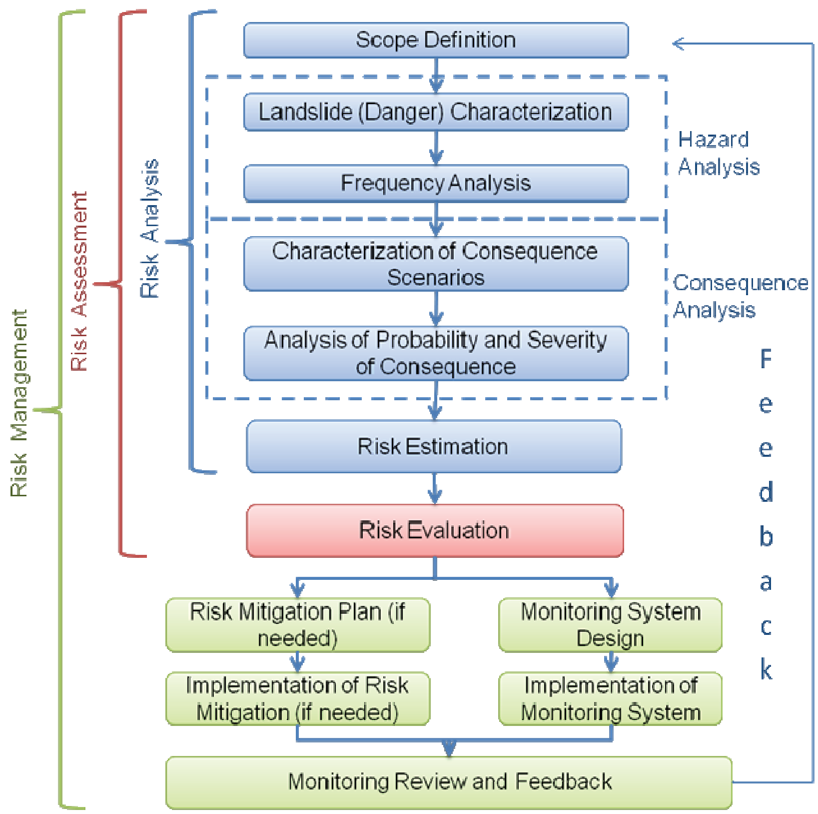


Figure 2-1 Generic risk management framework for natural and cut slopes. After Fell *et al.* (2005).

2.1. TERMINOLOGY

The terminology used in this study follows that developed by the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) Technical Committee on Risk Assessment and Management (TC32). The following are some of the most important terms and definitions as used in this study. To be consistent with the current state of practice the definitions of these terms are quoted from Fell *et al*. (2005):

> "Consequence: In relation to risk analysis, the outcome or result of a hazard being realised.
>
> Danger (Threat): The natural phenomenon that could lead to damage, described in terms of its geometry, mechanical and other characteristics. The danger can be an existing one (such as a creeping slope) or a potential one (such as a rockfall). The characterisation of a danger or threat does not include any forecasting.
>
> Elements at risk: Population, buildings and engineering works, infrastructure, environmental features and economic activities in the area affected by a hazard.
>
> Hazard: Population, buildings and engineering works, infrastructure, environmental features and economic activities in the area affected by a hazard.
>
> Individual risk to life: The increment of risk imposed on a particular individual by the existence of a hazard. This increment of risk is an addition to the background risk to life, which the person would live with on a daily basis if the facility did not exist.
>
> Risk: Measure of the probability and severity of an adverse effect to life, health, property, or the environment. Quantitatively, Risk = Hazard x Potential Worth of Loss. This can be also expressed as "Probability of an adverse event times the consequences if the event occurs".
>
> Risk mitigation: A selective application of appropriate techniques and management principles to reduce either likelihood of an occurrence or its adverse consequences, or both.
>
> Societal risk: The risk of widespread or large scale detriment from the realisation of a defined risk, the implication being that the consequence would be on such a scale as to provoke a socio/political response.
>
> Temporal (spatial) probability: The probability that the element at risk is in the area affected by the danger (threat) at the time of its occurrence.

> Vulnerability: The degree of loss to a given element or set of elements within the area affected by a hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss)."

2.2. QUANTITATIVE RISK ANALYSIS

The process of risk analysis, within the framework presented in Figure 1-1, and available methods that can be used for quantification of each step are provided by Fell and Hartford (1997), IUGS (1997), Einstein (1997), ERM (1998), Ho *et al*. (2000), Crozier and Glade (2005), Lee and Jones (2004), Leroi *et al*. (2005), Picarelli *et al*. (2005), AGS (2007). This section contains a brief overview of each step of the process and some discussions related to their quantification.

2.2.1. Scope Definition

There are some fundamental questions that need to be addressed when defining the scope of a QRA. These cover several subjects such as the amount and type of information required, the methodology best suited for analysis, the required resources, the units in which to measure the outcome of analysis, and what will the analysis (and in turn, the assessment) be used for (Crozier and Glade 2005).

The importance of defining a scope lies on the need to ensure that the relevant issues are being addressed, the needs of those implicated are being satisfied, and to avoid misunderstandings between parties (Fell *et al*. 2005).

In practice, technical, social, economic and political aspects dictate the scope of the assessment. The extent of the areas analysed, available resources, elements at risk of concern, time frame of interest, and level of detail, are typically determined by these factors (Crozier and Glade 2005).

2.2.2. Hazard Analysis

The hazard analysis consists of identifying the potential slope failures, characterizing them, and estimating their frequency or occurrence probability.

The analysis should be detailed and complete, following the scope defined in the previous step. The hazard analysis requires the contribution of several disciplines (geology, geotechnical engineering, hydrogeologists) (Amatruda *et al*. 2004a). The approaches adopted depend on the nature of the hazards (large landslide, regional shallow instabilities, rock falls along transportation corridors), the extent of analysis (a particular slope or a regional study), the availability of information (historical records), among others.

2.2.2.1. Slope Failure (Danger) Characterization

This step involves characterizing the potential slope failure in terms of type, size, velocity, location, travel distance, pre-failure deformations and mechanics (Picarelli *et al*. 2005). Understanding of the slope failure mechanisms and processes is fundamental for a proper characterization.

Several landslide classification systems that have been published can be used to determine the slope failure type (Varnes 1978, Cruden and Varnes 1996, Hutchinson 1988). The system proposed by Cruden and Varnes consists on building terms based on attributes such as state, distribution, style, rate of movement, water content, material and type of movement. Table 2-1 presents the abbreviated classification of slope movements proposed by Cruden and Varnes (1996).

Type of movement	Type of material		
		Engineering soils	
	Bedrock	Predominantly coarse	Predominantly fine
Fall	Rock fall	Debris fall	Earth fall
Topple	Rock topple	Debris topple	Earth topple
Slide	Rock slide	Debris slide	Earth slide
Spread	Rock spread	Debris spread	Earth spread
Flow	Rock flow	Debris flow	Earth flow

Table 2-1 Abbreviated classification of slope movements (After Cruden and Varnes 1996).

The size of a potential slope failure can be assessed through analysis of topography, geologic characteristics, deformation monitoring and numerical modelling. These approaches are applicable to hazards related to large slopes showing some signs of distress, or to analyses focused on specific locations. Under these circumstances, description of the location is concerned only with the extent of the deforming mass of the potential unstable slope, and an initial consideration of the areas likely to be impacted given failure occurs. The Turtle Mountain (Moreno and Froese 2009) and Checkerboard Creek slope in the Canadian Cordillera (Macciotta *et al*. 2010); the Cassas, Rosone, Oselitzenbach, Ceppo Morelli, Sedrun and Séchilienne landslides in Europe (IMIRILAND 2004); and the Shek Kip Mei slope in Hong Kong (El-Ramly *et al*. 2003); are some examples of site specific hazard characterizations for the purpose of hazard or risk analysis.

Location and size of slope failures in regional studies can be estimated following a stochastic approach, if historical records permit (Hungr *et al*. 1999, Dussauge-Peisser *et al*. 2002). Empirical and analytically derived relations between the onset of slope failure and ground characteristics (topography, geology, hydrogeology) are also used to assess the likely location of potential failures. This is typically presented in the form of susceptibility maps (Hunt 1992, Van Westen *et al*. 2003, Corominas *et al*. 2003, Blais-Stevens *et al*. 2012). Figure 2-2 presents the southern section of a debris flow susceptibility map for the Sea to Sky corridor (BC) developed by Blais-Stevens *et al*. (2012).

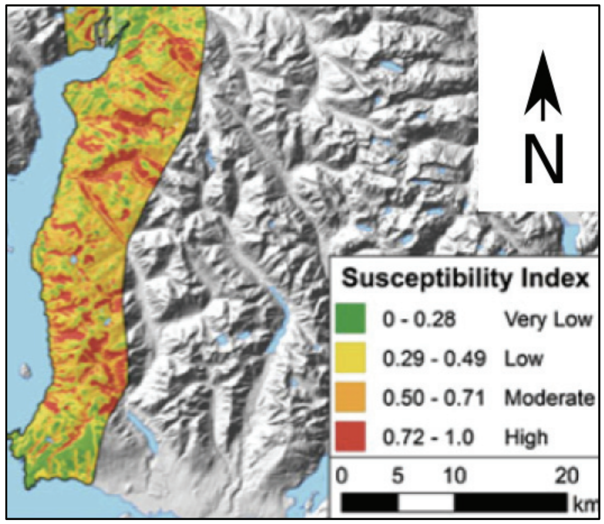


Figure 2-2 Example of a debris flow susceptibility map for the Sea to Sky corridor (BC) - southern area. After Blais-Stevens *et al*. (2012).

Pre-failure deformations include all movements that occur before a first time failure (Leroueil 2001). Within a natural slopes risk management framework, failure is defined as the sudden release of material, with such a volume and velocity, that the potential for a loss is present. In that regard, characterization of the slope in terms of its current deformation pattern and how this pattern might evolve before failure, becomes necessary. Discussions of pre-failure deformation patterns and the potential mechanisms behind them are presented by Zavodni (2000), Leroueil (2001), Rose and Hungr (2007). The importance of understanding the potential pre-failure deformations is reflected in the risk management stage. Here deformation monitoring and early warning systems, as a basis for the observational method, are commonly adopted for the mitigation of the risks of large landslides. Figure 2-3 shows the stages of slope movement as presented by Leroueil (2001).

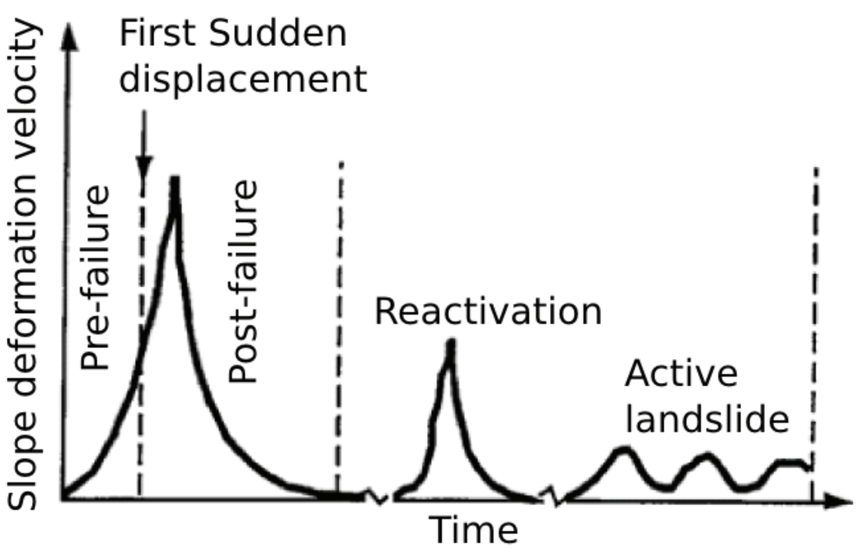


Figure 2-3 Different stages of slope movement (After Leroueil 2001)

As a minimum, a preliminary qualitative estimation of the potential travel distance of the failed material, as well as its velocity is necessary for this step. These estimates can be based on experience or empirical approaches. At this stage, only an estimate of the areas that can potentially be impacted by the slope failure is needed to assess the areal extent of the study. This aids in the identification of the elements at risk during the consequence analysis. detailed studies to estimate travel distance and velocity should be used, if available. These more detailed studies should be undertaken for the consequence analysis stage. Table 2-2 presents the landslide velocity scale proposed by Cruden and Varnes (1996), which is used in this study.

Velocity Class	Description	Velocity (mm/s)	Typical Velocity
7	Extremely Rapid		
		5×10^{3}	5 m/s
6	Very Rapid		
		5×10^{1}	3 m/min
5	Rapid		
		5×10^{-1}	1.8 m/h
4	Moderate		
		5×10^{-3}	13 m/month
3	Slow		
		5×10^{-5}	1.6 m/year
2	Very Slow		
		5×10^{-7}	16 mm/year
1	Extremely Slow		

Table 2-2 Landslide velocity scale (After Cruden and Varnes 1996).

2.2.2.2. Frequency Analysis or Slope Failure Probability

The metrics for expressing slope failure frequency or probability depend on the nature of the hazard being analysed (that is, slope failures at a regional scale or one - or a few - specific slopes). Following IUGS (1997), this frequency can be measured in terms of number of failures within the study area per unit of time (number of rockfalls per year along a transportation corridor) or the probability of a particular slope failing in certain period of time (10% probability of slope failure in the next 50 years). The former is usually what is referred to as failure frequency, while the later is referred to as failure probability.

The approaches available to estimate slope failure frequency or probability are discussed in IUGS (1997), Amatruda *et al*. (2004a), Lee and Jones (2004), Picarelli *et al*. (2005), Fell *et al*. (2005). These approaches can be summarized as follows:

Stochastic methods are used to derive slope failure frequencies. This can be done within the area of analysis or areas showing similar characteristics (geology, climate). Historic data can be in the form of recorded events and/or mapped ancient failures (large landslides) (Historical Approach). An example can be found in Hungr *et al*. (1999) for rock falls and rock slides along transportation corridors in British Columbia, Canada. Here, the relationship between slope failure volume and frequency is found for volumes less than 1 m^3 and up to 10^8 m^3. Bunce *et al*. (1997) estimated the annual rock fall frequency along a section of a highway in British Columbia by counting the number of scars left in the asphalt and their characteristics. Kong (2002) presents the average annual natural landslide frequency in Hong Kong based on historical records covering over 100 years.

Empirical methods correlate slope characteristics such as geology, morphology and vegetation, with the likelihood of failure. The outcome of such analyses is typically a susceptibility zonation. A combination with historic data and/or engineering judgment is necessary to obtain a value for the failure probability or frequency. Hantz *et al*. (2003) presents a novel approach combining geomechanical and historic data to develop a probabilistic approach to estimate rock slope failure frequencies. Other examples can be found in Guzzetti *et al*. (1999) and Coe *et al*. (2004).

Triggering event stochastic analyses correlate the occurrence of triggering events, such as rainfall storms, snow accumulation, wet seasons, seismic events; with the occurrence of slope failure. This method also utilizes historic failure data to obtain the correlation with the trigger event. The correlation can also be done through expert judgment. Examples of combining empirical methods with trigger analysis (rainfall events) and historic data are presented in Erener and Düzgün (2013), Kim *et al*. (1992) and Ko (2003).

Analytical solutions with trigger analyses use simple methods of stability analyses such as the infinite slope for shallow slope failures. These, in combination with slope characteristics such as overburden depth, slope inclination, and vegetation, are amenable for analysis using geographical information systems (GIS) software. When combined with trigger analyses such as seismic events and rainfall thresholds and their return periods, estimation of slope failure probabilities can be achieved. Software packages have been developed for this purpose, such as SHALSTAB (Montgomery and Dietrich 1994), SINMAP (Pack *et al*. 1998) and TRIGRS (Baum *et al*. 2002). Analytical solutions accounting for triggering factors can be applicable to large slopes if geological and geotechnical data in detail is available to build reliable models.

Historical approaches are based on previously observed slope failure events. It however uses historical records which span up to several hundred years. These records typically include large scale failures which society at the time deemed significant enough to be recorded. This method is more suitable to areas with a long history of record keeping such as Asian and European countries. This approach also uses ancient landslide recognition techniques to estimate return periods between large scale slope failures. Examples can be found in Forlati *et al*. (2004) for the Cassas landslide and Amatruda *et al*. (2004b) for the Rosone landslide.

Direct assessment of the failure probability through expert judgment is based on elicitation of subjective probabilities for a slope failure to occur. Tools such as event tree analysis and fault tree analysis are often used to break down the problem into logical steps leading to failure of the slope. Examples of these can be found in Bonnard *et al*. (2004) for the Sedrun landslide, Durville *et al*. (2004) for the Séchilienne landslide and Lacasse *et al*. (2008) for the Åknes landslide.

Probabilistic or Reliability methods consider the uncertainties in slope geometry, material properties, and failure mechanism, and the response to triggering events. This method requires knowledge of the ongoing slope processes, mechanisms and potential trigger characteristics. It is discussed in Nadim *et al*. (2005), El-Ramly (2001), and El-Ramly *et al*. (2002). Examples can be found in El-Ramly (2001, 2002) and Li *et al*. (2010).

It has to be noted that a combination of these seven approaches can also be adopted. Moreover, it is recommended that more than one method be used (Fell *et al*. 2005), although this might prove difficult for most projects. Table 2-3 presents the perceived suitability of these approaches for regional studies and single, large potential slope failures.

Approach	Regional scale / transportation corridor (typically widespread slope failures of limited volume and relatively high frequency such as debris flows and rock falls)	Local or single hazard (typically one potentially large slope failure)
Stochastic methods	Very suitable	Not suitable
Empirical methods	Very suitable	Not reliable
Triggering event stochastic analysis	Very suitable	Not suitable
Analytical solution and trigger analysis	Very suitable	Very suitable approach if all necessary data are available
Historical approach	Very suitable	Has been applied but should be recognized it does not consider the particularities of the hazard being analysed relative to the context of the historical data
Direct assessment of the failure probability through expert judgment	Applicable only if all other suitable methods cannot be applied due to lack of data	Sometimes the only applicable approach
Probabilistic or Reliability methods	Not practical	Best approach if all necessary data are available

Table 2-3 Approaches to estimate slope failure probabilities and their suitability for regional studies and single, large potential slope failures.

2.2.2.3. Secondary Hazards

Slope failures can also produce secondary effects that may result in greater losses than those caused by direct impact of the failed material (Lee and Jones 2004).

Slope failures in mountainous areas can potentially block narrow river valleys, forming natural dams. Schuster (1986) and Sassa (1999) present some information and discussion regarding historic landslide dams. Landslide dams in turn cause flooding of the upstream river valley, potentially impacting developed areas and economic activities such as agriculture. Raises in water levels can also lead to erosion and elevated water pressures at the toe of the valley slopes, decreasing their stability.

Overtopping and erosion of the poorly consolidated landslide dam can lead to breaching of the dam and flooding of downstream areas (Lee and Jones 2004). This process is typically unexpected and water is released violently (Korup 2002). The effects from such occurrences have the potential to be catastrophic. Failure of the Deixi landslide dam in China in 1933 (Li Tianchi *et al*. 1986) and the Tunawaea landslide dam in the North Island of New Zealand in 1992 (Webby and Jennings 1994) are two examples. Korup (2002) also presents a literature review on landslide dams and their potential effects.

Fast moving landslides entering bodies of water, such as lakes and reservoirs, have the potential to generate large impulse waves that can overtop/breach dams and create catastrophic flooding downstream (Lee and Jones 2004, Glade and Crozier 2005). The Vaiont landslide in 1963 (Semenza and Ghirotti 2000) generated a landslide impulse wave within a reservoir which overtopped the dam and flooded a village downstream causing over 2000 fatalities. Landslides in Norway generating impulse waves have been responsible for several lives being lost as consequence. Tjelleskredet in 1756, Loen in 1905 and 1936, and Tafjord in 1934, claimed the lives of 32, 61, 73 and 41 people, respectively (Lacasse and Nadim 2007). The Lituya Bay tsunami in Alaska (1958) was triggered by an earthquake generated landslide (Mader and Gittings 2002) which inundated the shoreline of the bay with a run-up of about 530 m. Fortunately, because of the remoteness of the area, no fatalities were reported.

Other secondary hazards can include deforestation of large areas and the loss of pedological soils (Glade and Crozier 2005). This can affect the economic activities in the area, and enhance erosion of the slopes. Also, other economic, societal, environmental and political consequences need to be considered which depend on the particular context where the slope failure occurs.

2.2.3. Consequence Analysis

Given a slope failure occurs, the outcome of the failure needs to be assessed. This assessment should consider not only the direct impact of the landslide event, but any foreseen secondary hazards.

Typically, consequences are measured in terms of property damage (monetary units) and loss of life. However, these may include consequential losses (economic activities or transportation corridors being interrupted), reputation of the owners and engineers, loss of resources and heritage, litigation costs and potential criminal charges, adverse social, political and environmental effects, and cultural losses (Lee and Jones 2004, Fell *et al*. 2005).

Some approaches that can be adopted to estimate the consequences of a slope failure, estimate the consequences directly based on experience, historical records for similar contexts, and judgment. Other approaches rely on rational frameworks based on consideration of key factors affecting the outcome of slope failure (Wong *et al*. 1997, Lee and Jones 2004). Estimating consequences directly from experience and expert judgment is typically adopted where the scenario components are too complex to be considered systematically and where past experience permits a sensible judgment to be made. This approach, however, is best suited to qualitative assessments. Using historical records has the advantage of utilizing real case scenarios with measured consequences to predict consequences of potential slope failures. However, extrapolation to new situations can be cumbersome given differences in the hazard characteristics and the context of the elements at risk. In any case, historic records are useful for populating consequence models built for the purpose of consequence analyses following approaches based on rational frameworks.

Consequence models provide this rational framework to estimate the effects of a potential slope failure. The assessment is focused on scenarios and scenario components judged to be relevant to the particular hazard (Wong *et al*. 1997) and follows a cause-effect logic. In this approach the consequence analysis usually consists of: 1) identification and quantification of the elements at risk, 2) evaluation of the value of the elements at risk, 3) Estimation of the temporal and spatial probabilities for the elements at risk, 4) Estimating the vulnerability of the elements at risk. The consequences are then calculated as the product of the value of the elements at risk, the temporal and spatial probabilities and their vulnerability (Amatruda *et al*. 2004a, Fell *et al*. 2005). Wong *et al*. (1997) further distinguish event tree approaches from consequence models, however the former can be seen as a subset of the later. Figure 2-4 presents an overview of the event tree analysis for estimation of the consequences on a railway operation given a landslide occurs (Bunce 2008). It is clear that comprehensive analyses can become complex, and event trees aid in the visualization of the consequence model.

2.2.3.1. Identification and Quantification of the Elements at Risk

Elements at risk can be the population, buildings, infrastructure, vehicles, economic activities and environmental features that can be affected by a slope failure (Fell *et al*. 2005). Elements at risk are identified and classified based on estimations of the potential areas affected by the slope failure (Amatruda *et al*. 2004a). The elements at risk are then counted (number of buildings, population, length of highway affected), or estimated (inhabitants per apartment building floor, number of passengers per vehicle type), depending on the scenarios analysed.

The elements at risk can be monetarily quantified (Cost of replacement or repair for buildings, infrastructure, suspension of economic activity) in order to quantify the consequence (expected impact value). Quantification can also be expressed as number of elements lost and their probability (Amatruda *et al*. 2004a). When it comes to consideration of the consequences in a comprehensive manner, assigning monetary value to the losses of different types of elements at risk aids in adding their consequences into a global consequence value. Assigning value to the elements at risk can be done by: 1) computation of specific values for each element at risk, 2) the use of utility functions relating the degree (or magnitude) of the loss to monetary value, 3) through empirical formulas, or 4) expert assessment of the global value of a certain area (Amatruda *et al*. 2004a).

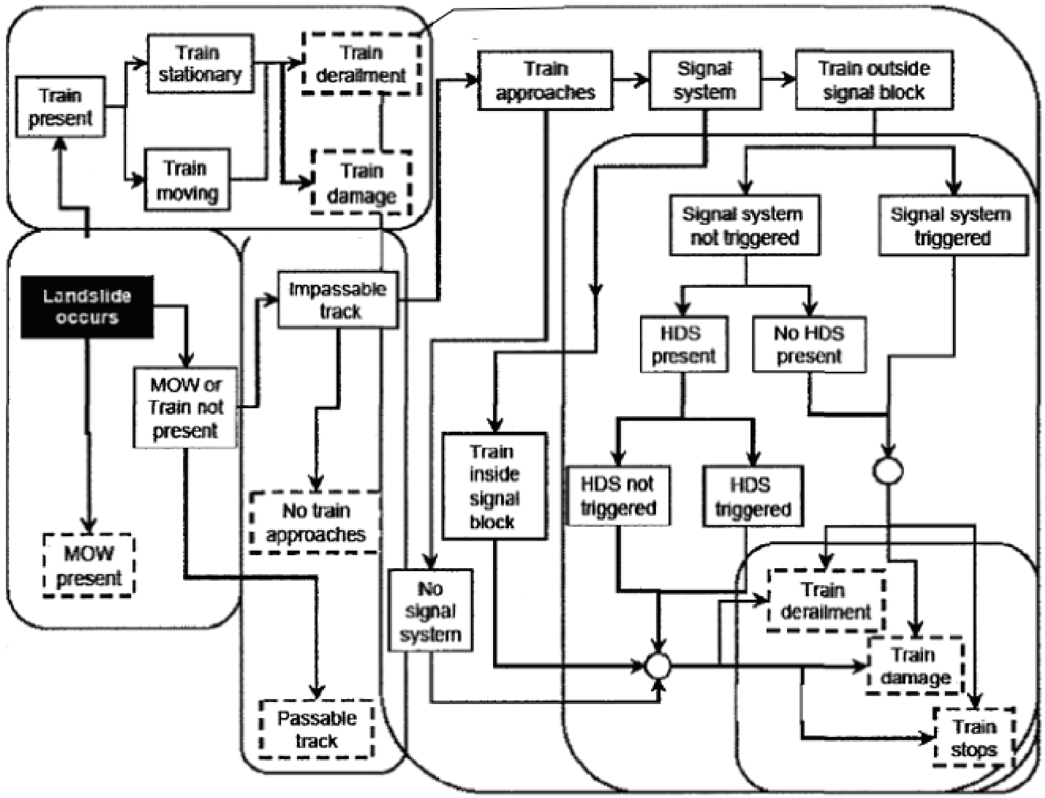


Figure 2-4 Example of an event tree analysis for estimation of the consequences in a railway operation given a landslide occurs After Bunce (2008). MOW: maintenance of way crews. HDS: hazard detection system.

When it comes to quantification of lives being lost after a slope failure, assigning a monetary value to life would aid in combining this consequence with the loss of other elements at risk. Some estimates of the economic value of life have been proposed for different contexts, mainly considering people as a resource in an economic activity, however the approach conflicts with ethical traditions (Skjong 2002). Quantification of the population at risk is typically done by estimating the number of people exposed and their relation with the hazard (amount of exposed general public or number of workers which obtain benefit from being exposed to the hazard).

2.2.3.2. Temporal and Spatial Probability

Temporal and spatial probability quantifies the potential for intersection in space and time between the hazard, and the elements at risk. The probability of spatial intersection depends on the probability that the moving mass overlaps laterally with the element at risk, and the probability that it travels as far as the location of the element at risk (Roberds 2005). These can be calculated geometrically, however, unless the elements at risk are located on top of the failing slope, there will be uncertainty regarding the characteristics of the displaced materials and their intersection with the elements at risk. This can be dealt with by adopting different estimation techniques and applying probabilistic approaches.

Methods for estimating the travel distance of the failed material can fall into the empirical or analytical groups. Empirical approaches commonly used are based on geomorphologic or geometrical observations of slope failures (Hungr *et al*. 2005). In these approaches the travel distance of the failed mass is related to characteristics of the pre-failure slope configuration. An example of these are the equations presented by Finlay *et al*. (1999) and Hunter and Fell (2003) relating travel distance to slope height, landslide volume, slope angle and landslide width. Other authors (Heim 1932, Corominas 1996, Hungr 1990) have analysed the angle with the horizontal of the line connecting the highest point of a landslide scarp to the tip of the displaced material (angle of reach, also known as reach angle or travel angle or travel distance angle). Some present findings on the volume-dependency of these relations (Scheidegger 1973, Corominas 1996). Table 2-4 shows the results of regression analyses for the tangent of the angle of reach (H/L = path height / path length) considering landslide volume (Corominas 1996).

Landslide Type	No. Events	A	B	r^2
All Landslides	204	-0.047	-0.085	0.625
Rockfalls				
All	47	0.21	-0.109	0.759
Obstructed	16	0.231	-0.091	0.834
Deflected	6	1.078	-0.233	0.854
Unobstructed	14	0.167	-0.199	0.924
Transitional Slides				
All	69	-0.159	-0.068	0.670
Obstructed	23	-0.133	-0.057	0.756
Unobstructed	42	-0.143	-0.080	0.796
Debris Flows				
All	71	-0.012	-0.105	0.763
Obstructed	29	-0.049	-0.108	0.849
Channelized	19	-0.077	-0.109	0.690
Unobstructed	18	-0.031	-0.102	0.868
Earthflows and Mudslides				
All	17	-0.214	-0.070	0.648
Unobstructed	8	-0.220	-0.138	0.908

Table 2-4 Regression analyses for the tangent of the angle of reach (H/L) considering landslide volume (Corominas 1996). log(H/L) = A + B log(volume).

The results in Table 2-4 show fair correlations for run out prediction purposes. They can be used to evaluate the likelihood of intersection with the elements at risk, not without the use of some engineering judgment. More refined analytical methods can be adopted. These, however, need to be validated (calibrated) for the uncertainty in the results to be lower than those obtained using the regression results in Table 2-4 or other empirical methods.

Analytical methods aim to model the failed material based on the physical mechanisms of solid and fluid dynamics (Hungr *et al*. 2005). Some of these models are solved through numerical simulations (finite difference methods, particle flow codes) and are either two or three-dimensional in their treatment of the flow of the failed material. Examples of two-dimensional models are found in Hungr (1995), Sousa and Voight (1991), and McDougall and Hungr (2005). Examples of three-dimensional models are found in Sassa (1988), O'Brien *et al*. (1993), and McDougall and Hungr (2004). Some analytical methods are based on energy considerations that include different formulations based on lumped mass approaches such as the ones by Sassa (1985) and Hutchinson (1986), consideration of the rolling friction mechanisms (Huang and Wang 1988), and consideration of momentum transfer (Van Gassen and Cruden 1989). When properly calibrated, analytical methods can provide more insight into the potential consequences than empirical methods. Both the empirical and analytical approaches presented here, as well as other studies are discussed in Wong *et al*. (1997) and in some detail in Hungr *et al*. (2005).

Intersection of stationary elements at risk, such as buildings and infrastructure, with the hazard only depends on the spatial intersection. Consequences on non-stationary elements at risk (people living in the buildings or using the infrastructure, and vehicles through transportation corridors) also depend on their temporal intersection with the hazard. Temporal aspects for the estimation of slope failure consequences can be separated in temporal aspects of the hazard and temporal aspects of the elements at risk (Roberds, 2005). If these are non-correlated, the temporal probability can be estimated by simple probabilistic algebra. As an example, earthquake-triggered slope failures can be assessed to have the same occurrence probability during the day and during the night (given a seismic event occurs there is a 50% probability that it occurs in the day and 50% it occurs in the night). When analysing the risk to apartment buildings located in the path of the potential debris run-out, the hazard probability can then be combined with the probability that individuals are at home during the night (high probability - having dinner, sleeping) and during the day (low probability - maybe working 5 out of 7 days).

However, it is common for the temporal probability of the hazard and the elements at risk to be correlated. In the previous example, if the trigger is linked to precipitation, the temporal aspect of the elements at risk is also affected. People tend to be indoors during heavy precipitation events, and the temporal intersection probability increases. Even when this situation can also be analysed using probability theory, a previous step is necessary which involves understanding of the hazard mechanisms and the behaviour of the elements at risk in more detail than for the previous scenario. This dependency can easily become more complex if for example it is recognized that in the area, precipitation events occur during the later hours of the evening with a higher probability than during other periods of the day. This can be combined not only with the probability of people being indoors, but also with people sleeping, which can be assessed to represent a higher degree of vulnerability. Now, if the area is heavily visited for alpine activities during the winter months, when precipitation patterns are different than for other periods of the year, another degree of complexity adds to the analysis.

Another consideration for these temporal aspects are the changes through time of both the hazard and the elements at risk. For example, continuous slope deformation can lead to a more unstable state. The trigger needed for the onset of sudden failure might then be of lesser intensity, and have a shorter return period. The temporal aspects of the elements at risk can also change with inhabitants' life style, and changes in demographic and building codes (Lee and Jones 2004, Roberds 2005).

2.2.3.3. Vulnerability of the Elements at Risk

Some definitions of vulnerability include the probability and severity of landslide (or secondary hazard) impact and the level of damage the elements at risk can survive given they are impacted (Roberds 2005, Alexander 2005). Following IUGS (1997) and Fell *et al*. (2005), vulnerability is defined in this research as the degree of loss given the element at risk is impacted by the failed material or secondary hazard. It is evaluated as a conditional probability given the element is impacted and is expressed on a scale of 0 (no loss) to 1 (total loss).

The factors that most affect vulnerability to direct landslide impacts are the volume and speed of the displaced material, the location of the element at risk and structure types (both for people protection and for degree of damage of the structure) (Fell *et al*. 2005). As these factors are widely different between landslide case histories, and given the uncertainties in predicting post failure behaviour, vulnerability estimations are usually based on expert opinion and are embedded in great uncertainty. This is illustrated by significant variations in the fatality rates reported after landslides (Alexander 2005).

One approach for expressing vulnerability is the use of damage functions. These express the degree of loss (or damage) as a function of the hazard characteristics (failure type, volume and velocity), and the characteristics of the elements at risk (structure type, if people are indoors or outdoors). Examples of these developed for flood hazards can be found in Brown and Graham (1988), DeKay and McClelland (1993), Graham (1999), McClelland and Bowles (2002), and Peng and Zhang (2012). Unfortunately, damage functions have generally not been developed for landslide intersections (Roberds 2005). It is to expect, however, that given all the variables involved in landslide interactions with the elements at risk, damage functions need to be case specific and based on previous case histories and expert opinion considering reasonable assumptions.

Examples of vulnerability values adopted for landslide risk analyses can be found in Wong *et al*. (1997), Bunce *et al*. (1997), Amatruda *et al*. (2004c), Bonnard *et al*. (2004), Kong (2002), Hungr *et al*. (1999), Li *et al*. (2010), Mostyn and Sullivan (2002), El-Ramly *et al*. (2003) and Bunce (2008). El-Ramly *et al*. (2003) estimated, based on judgment, a fatality rate of 1 given a total building collapse and 0.6 if the debris enters the ground floor, but without total building collapse.

Regarding secondary hazards, Bunce (2008) estimated from historic records that the probability of a fatal accident following a main line freight derailment was about 1.3%.

2.2.4. Risk Estimation

The calculation of risk is the mathematical combination of the hazard assessment (probability of slope failure), and the consequence assessment (number of elements at risk, spatial and temporal probabilities, vulnerability, and value of the elements at risk). It can be done through probabilistic algebra, reliability methods, or simulation (IUGS 1997). It can be expressed as:

$$\text{Risk} = \sum P_{[H]} \times P_{[T]} \times P_{[S]} \times V_{[E]} \times E$$

where $P_{[H]}$ is the slope failure probability, $P_{[T]}$ and $P_{[S]}$ are the temporal and spatial probability of intersection, $V_{[E]}$ is the vulnerability of the element at risk, and E is the economic value of the element at risk (in the case of buildings, infrastructures, economic activities) or the number of people exposed (when estimating risk to life). To obtain the total value of risk, summation is required over all hazards and for all exposed population.

The estimated risk can be presented in several ways. The most commonly adopted are 1) the annual risk, where each hazard is combined with their consequences and summed over all potential hazards. It is expressed as expected economic loss per year or potential loss of lives per year; 2) frequency - consequence pairs, where the annual probability of certain magnitudes of loss are plotted (the probability of 1 life, 5 lives or 100 lives being lost); and 3) cumulative frequency - consequence plots, where the probability of a consequence magnitude or higher is plotted (Fell *et al*. 2005).

The units selected for estimation should consider the need for communicating risks to non-specialists (use comprehensible units), and the need for being compared to selected criteria and other risks (compatible with the state of practice) (Lee and Jones 2004). Compatibility with the intended use of the analysis is of most importance (whether for public safety reasons or feasibility of new projects).

2.3. QUANTITATIVE RISK ASSESSMENT

The quantitative risk assessment step is accomplished when the estimated risks are evaluated (compared) against value judgments and tolerance (or acceptability) criteria (Fell *et al*. 2005). It aims to provide objective advice to decide if mitigation of these risks need to be considered, and the urgency of these mitigation measures (Lee and Jones 2004). The process requires making judgments considering political, legal, environmental, regulatory and societal factors.

Evaluation of risk can be done in monetary units. This is particularly straight forward when applied to buildings and infrastructure, or economic activities.

Value judgment risk thresholds are typically defined by the owners and regulators based on economic impact. Cost-benefit analyses are also applied to assess if mitigation measures are worth implementing.

Some aspects of adverse consequences, however, are not easily expressed in monetary units. Examples of these are amenities, the environment, cultural heritage and life (Lee and Jones 2004). Even when there are some techniques available for estimation of the economic value of these aspects, public perception is the main driver for definition of any evaluation criterion. Particularly for the case of human lives, the value of a statistical life has been proposed to aim in evaluating potential loss of life in monetary units (Rice 1966, Lave and Seskin 1970). This approach, however, is considered unethical by a portion of the population (Skjong 2002). There is the tendency in the geotechnical community to evaluate risk to life in terms of the probability of an individual being killed (individual risk) and the cumulative probability of a certain number of people or more being killed (societal risk) (Ho *et al*. 2000, Fell *et al*. 2005, Leroi *et al*. 2005).

Some examples of risk to life evaluation criteria are those by the Health and Safety Executive (HSE) in the United Kingdom for land use planning around industries (HSE 1988, 1992 and 2001), the Australian National Committee on Large Dams (ANCOLD) for population exposed to potential dam failures (ANCOLD 2003), the United States Bureau of Reclamation, also for people exposed to potential dam failures (U.S. Bureau of Reclamation 2003); and the Hong Kong Special Administrative Region Government for land development in landslide prone areas (ERM 1998). These criteria have also been discussed in Finlay and Fell (1997), Ho *et al*. (2000), Leroi *et al*. (2005), Ale (2005), Porter *et al*. (2009) and Scarlett *et al*. (2011), among others. An example of the societal risk to life evaluation criterion applied in Hong Kong is presented in Figure 2-5 (ERM 1998). The individual risk to life tolerance thresholds in Hong Kong are set at 10^{-4} for existing situations (existing development and existing slopes), and10^{-5} for proposed development. The reader is referred to Chapter 5 of this thesis for details on how to develop criteria (or validate previously proposed criteria) for a specific project.

2.4. RISK MANAGEMENT

Conceptually, the full range of procedures and tasks that lead to the implementation of policies and risk mitigation strategies are collectively referred to as risk management (Crozier and Glade 2005). This concept is applicable to any approach adopted to deal with uncertainty and risk in geotechnical practice. It is noted that the use of more than one approach (factors of safety, reliability-based design, qualitative risk rankings, QRA) can be part (and usually is necessary) of a comprehensive risk management strategy.

Within the framework presented in Figure 1-1, after the risk levels have been evaluated (and considered acceptable, tolerable or intolerable) a decision needs to be made regarding appropriate approaches for risk control. These can include risk reduction works (or risk mitigation - in case risks are deemed to need further reduction) and monitoring (both to assess changes in the risk levels and as part of risk mitigation when linked to early warning systems). When risk control is implemented following a risk assessment, and feed back is given to earlier stages of the process, the risk management framework is completed (Ho *et al*. 2000). In this regard, an assessment of the new risk levels is necessary after risk control is implemented.

The decisions whether or not to reduce risk, and which approaches for risk mitigation to adopt involve consideration of a range of factors such as technical feasibility, economic viability, environmental acceptability and socio-political considerations (Lee and Jones 2004). Mitigation options can include (Fell *et al*. 2005, Leroi *et al*. 2005):

- Avoid the risk (walk off strategy),
- Slope stabilization measures such as drainage and buttressing to reduce the probability of slope failure,
- Construction of catch fences for rock falls or catch dams for debris flows to reduce the spatial probability of the hazard reaching the elements at risk,
- Monitoring and early warning to reduce the temporal probability that the elements at risk be present when the hazard is realized,
- Transfer the risk, and compensating for the increase in risk levels,
- Increase the level of knowledge through increased investigation and analyses. This is usually temporary and aims to reduce uncertainty, improving the efficiency in decision making regarding resource allocation, and;
- Public information and involvement. Typically necessary to reduce exposure to the hazard and increase the efficacy of early warning systems.

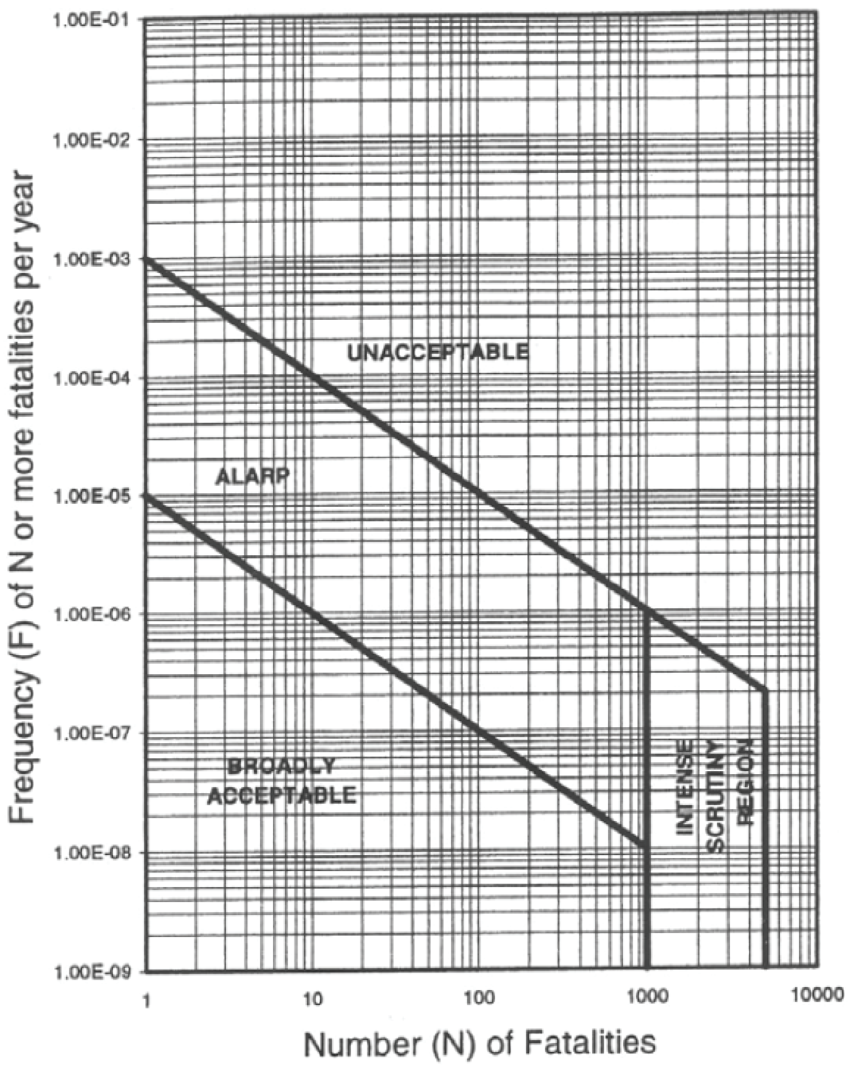


Figure 2-5 Societal risk evaluation criterion for development in landslide prone areas in Hong Kong (After ERM 1998). ALARP: As Low As Reasonably Practical.

Analysis and evaluation of risk after mitigation and monitoring is in place makes risk management an iterative process. The need for updating this process given the changing contexts and in light of newly acquired information (through monitoring, ongoing studies, increase in experience from other cases) makes it also a continuous process.

2.5. NECESSARY CONCEPTS FOR THE FOLLOWING CHAPTERS

The main concepts, state of practice and tools available for QRA have been presented in this Chapter. Process modelling and Event Tree Analysis (ETA) is applied in this research to model the sequence of events leading to a loss (and estimate the levels of risk). The QRA case studies presented are focused on quantifying how uncertainty in the input variables needed for risk estimation is carried on through the analysis. This uncertainty will be reflected as uncertainty in the estimated values of risk. Some input variables will show greater uncertainty depending on the information available and our knowledge of the particular subject. Population of these in the process model will rely significantly on expert opinions and subjective probabilities. This section presents some concepts and available tools to deal with elicitation of expert opinions and methods of probabilistic analyses that incorporate measures of uncertainty. The section also includes a brief presentation of other types of uncertainty not dealt with in this research but that need to be considered by the practitioner, at least in a qualitative or conceptual manner.

2.5.1. Expert elicitation methods and expert aggregation approaches

2.5.1.1. Expert Elicitation

An expert opinion can be defined as a formal judgment, or belief, of an expert on a particular matter, which is subjective, and typically based on uncertain information or knowledge (Ayyub, 2001). Expert opinion is part of every engineering and risk management decision. However, as the uncertainties become greater than the knowledge of the system analysed, structured methods for expert elicitation become a necessity.

One of the most common methods for expert elicitation is the Delphi method (Helmer, 1968). it consists of the following steps (after Ayyub 2001):

- Definition of the information required from the elicitation process,
- Development of questionnaires to gather the information,
- Selection of experts,
- Familiarization of experts about the subject matter to be elicited,
- Elicitation of experts - gather responses to the questionnaires,
- Aggregation and presentation of results,
- Review of results by the experts and revision of initial responses. Responses outside the 25% and 75% percentile values should be accompanied by proper justification,
- Repetition of previous three steps until consensus is achieved or results are considered precise enough, and;
- Summary of results and justification.

Variations of this method typically consist of the type of questionnaire (direct, indirect or parametric), the level of interaction between experts, and how results from different experts are aggregated.

The questionnaires can be developed such that the elicitation process is direct, indirect or a parametric estimation that deals with uncertainty (Ayyub 2001). The direct method elicits a direct estimate of the belief of an expert in the information required from the process. Indirect elicitation consists in eliciting answers on metrics more familiar to the experts, and later deriving the required metrics for analysis. An example of this would be eliciting the time to failure of a system and then deriving the failure probabilities from it. Another approach is to build questionnaires comparing the unknown required information with familiar situations with known answers. Answers are in the form of relative measures with respect to the familiar situations for which actual statistical or measured values exist.

A parametric estimation obtains a measure of the information required and a measure of the confidence interval. An initial step consists in eliciting the information (such as the median estimate of a probability), followed by a second step where a measure of dispersion is elicited. These elicited values can then be used to compute confidence bounds for the elicited information (Preyssl and Cooke 1989, Ayyub 2001).

There can be different levels of interaction among experts. The nominal group technique discussed in Morgan and Henrion (1992) includes discussion among experts after the initial elicitation responses are presented.

2.5.1.2. Aggregation of Expert Elicitation Responses

Consensus can be reached through discussion among the elicited panel of experts. This can include measures of uncertainty such as confidence intervals or outer quartile values. The shortcomings rely in the potential for conformity within the group, dominance of strong minded participants and biases due to common background (Ayyub 2001).

Aggregation can also be done mathematically. Responses can be treated as a statistical sample and point estimates derived from them (average, median, percentiles). There will be situations where different experts might provide responses with different degrees of reliability on their answers. A scoring of experts can then be applied to weight the responses during aggregation. This process can consist in self-scoring or each participant scoring the other members of the panel.

Methods for weighted combinations of opinions are summarized in French (1985), Genest and Zidek (1986) and Ayyub (2001). These include weighted arithmetic, geometric and harmonic averages. The statistical sample of responses can also be increased according to each expert score and percentiles be derived from the modified sample.

Ayyub (2001) also describes three methods for expert elicitation aggregation based on uncertainty measures. These can deal with various uncertainty types, offering a strengthened approach. On the other hand, they are of an increased complexity and computational demand.

The uncertainty invariance criterion facilitates the combination of various uncertainty measures by carefully constructing them in terms of compatible scale and units. Then, these uncertainties can be directly combined. No further discussion is presented here regarding this approach and the reader interested should see its application in Brown (1980).

The minimum uncertainty criterion focuses on finding the tendency of the responses, in order to maximize information retention, thus minimizes uncertainty. This can lead to a reliance on the response tendency as representative of the true value.

The maximum uncertainty criterion focuses on utilizing all the information gathered, both the tendency of responses and the spread (or uncertainty). Likelihood distribution of responses can be built given a set of constraints based on the elicited responses. The maximum uncertainty criterion would then rely on probability distributions showing maximum entropy (uniform distribution if maximum and minimum values are selected as constraints, normal distribution if an expected value and variance are selected as constraints).

Aggregation can also be done by defining intervals for the elicited responses. These can then be treated as fuzzy sets, where the intervals are a special case. Fuzzy arithmetic and calculus can then be applied to combine expert opinions in this approach (Ayyub 2001).

In this research study, required or missing information was elicited to populate the input variables of the models developed to estimate the risk levels for the case studies presented. Elicitation was done directly for the intervals believed to represent the range of realistic values the elicited information can adopt. Uncertainty was further modelled by defining a uniform probability distribution for each interval (maximum uncertainty criterion). This approach allows for the uncertainty in the estimated risk to be evaluated given the uncertainties in the elicited information.

2.5.2. Methods of Probabilistic Analysis Incorporating Measures of Uncertainty

Incomplete information, or lack of it, leading to significant input of expert opinions for risk estimation involves dealing with uncertainty related to the model input parameters, that will carry on and be reflected as uncertainty in the estimated risk.

This subsection presents a brief summary of some methods available for probabilistic analysis which incorporate measures of uncertainty. The aim is to obtain a value representing the risk levels of the system being analysed (central tendency) and an estimation of the uncertainty related to this value.

2.5.2.1. Analytical Methods

Analytical methods can be derived where probability density functions (PDF) of the input variables are integrated to derive a mathematical expression for the probability density function of the estimated risk. Given the significant mathematical complexities of this approach, risk estimations require overly simplified models, and comprehensive analysis become impractical.

2.5.2.2. *Approximate Methods*

Approximate methods obtain an approximate value of the point estimates of the risk level PDF based on point estimates of the input variables PDFs. The most common methods are the First Order Second Moment (FOSM) and variants, and the Point Estimate method. These require a mathematical expression to estimate risk, generically named performance function.

The FOSM method replaces the direct integration of the input variables PDFs by their Taylor's series expansion about the mean, truncated at the linear terms. The mean is approximated by solving the performance function for the mean values of the input variables. The variance is estimated as the sum of the performance function partial derivates multiplied by the covariance for all pairs of input variables. Detailed description of this method is available in text books such as Ang and Tang (1984) and Harr (1987). The accuracy in the approximation decreases as the non-linearity of the performance function increases. An example applied to risk analysis can be found in Cassidy *et al*. (2008). They estimated the risks related to a submarine landslide in Norway and the variability in the results by means of FOSM approximation.

The point estimate method consists in replacing the continuous PDF of each input variable with a two-value discrete distribution such that the mean and variance are the same. Estimates of risk are then evaluated for all possible combinations of the two-value discrete distributions of all input variables and the mean and variance obtained. Details on this method can be found in Rosenblueth (1975, 1981) and Harr (1987).

The advantage of these methods reside in their ability to reduce the computational requirements, thus providing an efficient approach to approximate the point estimates of the estimated risk PDF. However, when the output PDFs are far from a normal distribution (often the case in risk estimates covering over two orders of magnitude), these point estimates do not give a clear picture of the actual risk probability distribution and could be hiding important information to the analyst. This is illustrated in Chapter 3 when discussing the results of the analysis.

Another example using approximate methods in risk analysis can be found in You and Tonon (2012). They highlighted the uncertainty related to defining PDF of variables where limited or no information is available. To deal with this uncertainty, they proposed the use of imprecise probabilities within ETA. Imprecise probability constructs a general convex set of probability distributions. Random sets, normalized fuzzy sets, and envelopes of cumulative probability distributions are special cases of imprecise probabilities (You and Tonon 2012). This method then renders a lower and upper estimate of risk based on approximation techniques.

2.5.2.3. *Monte Carlo Simulation Techniques*

Monte Carlo simulation techniques consist of sampling processes over the input PDFs in order to populate the model developed to estimate risk. Values of the input variables are selected on the basis of random number generation and variable mapping according to their cumulative probability distribution. Each run of the simulation, or iteration, is statistically treated as an observation within a set of possible outcomes. As the number of iterations increase, the outcome probability distribution resembles more and more that of the entire set of possible outcomes. This approach allows for a PDF to be fitted to the estimated risk. Different Monte Carlo simulation techniques mainly differ in the algorithms used for random number generation or in techniques developed to minimize the number of iterations and reduce computational effort. The recent developments in computational capabilities, however, allow to run a large number of iterations within minutes even for complex models. Details of Monte Carlo simulation techniques for reliability analysis are presented in Andrews and Moss (2002) and Ayyub (2003). Monte Carlo simulation is used in this research study.

2.5.3. Other types of Uncertainty not Dealt With in this Research

A simple categorization of uncertainty in geotechnical engineering was presented by Morgenstern (1995). The three categories presented were: 1) parameter uncertainty; 2) model uncertainty, and; 3) human uncertainty. A detailed discussion of these can be found in El-Ramly (2001).

2.5.3.1. Parameter Uncertainty

This type of uncertainty is related to the input parameters and can be attributed to data scatter and systematic error. Data scatter can arise from real variability of the inputs or from random error when performing observations (random testing error, accuracy of observation approaches). This type of uncertainty tends to populate the observations around central values that can be argued to be the representative or true values. The methodology presented in this study directly addresses this type of uncertainty given that enough measures of the input variable exist.

Systematic error tends to shift the central tendency (or average values) observed. This can arise from statistical error (not enough observations) or from measurement bias. When defining values for the input variables for risk estimations it is common to rely on limited information complemented by expert opinion. This study addresses this issue by defining upper and lower values of the input variable, and defining a PDF between them which reflects the amount of available information and associated uncertainty. However, elicitation of these upper and lower values, as well as defining the shape of the PDF, heavily rely on expert opinion which is susceptible to bias. This bias can be reduced by expert aggregation approaches, however this study does not particularly address the issue.

2.5.3.2. Model Uncertainty

Model uncertainty is related to the differences between reality and the theory developed to describe an issue, any empirical relationships built, and the necessary simplifications of the adopted approach. These uncertainties exist at different levels within the QRA framework. Model uncertainty related to particular analyses to estimate the values for the input parameters can be accounted for, to a certain extent, by a conservative definition of the upper and lower values of the input variable and the shape of the PDF. However, the uncertainty related to the process model developed for risk estimation can only be reduced by expert aggregation and peer review. A higher level of model uncertainty is related to the overall evaluation of risk, where issues related to the existence of other risks not being aggregated, unforeseen events or consequences, and discarded events are not appropriately accounted for in the analysis. This is not treated in this study.

2.5.3.3. Human Uncertainty

Human uncertainty is related to the potential for human errors. This uncertainty can be present in the analysis itself, however proper expert aggregation, peer reviews, and the use of a clear and structured framework should minimize this to negligible levels. On the other hand, human uncertainty becomes difficult to address when the value of input variables depend on the response of individuals to the situation under analysis. This response will be influenced by the knowledge level of the individuals, the efficiency of the risk communication strategies, the emergency protocols in place, and the physical environment (weather, time of day), among others.

The importance of this human uncertainty has lead to the development of a framework to deal with human reliability (Dhillon 1986, Dougherty and Fragola 1988, Ayyub 2003). However the uniqueness of the situations typically dealt with in geotechnical engineering create difficulties in identifying potential human errors and assessing their probabilities (El-Ramly 2001). This type of uncertainty is not dealt with in this study.

CHAPTER 3: QRA OF INSTABILITIES IN ROCK SLOPE CUTS ALONG A SECTION OF A RAILWAY

The section between miles 2 through 15 of the Canadian Pacific Railway (CP) Cascade subdivision is considered a highly hazardous, intensively risk-mitigated section (Macciotta el at. 2011). As such, it was selected as a case study for a quantitative risk assessment (QRA) to be performed. Given the nature of rock fall phenomenon and the characteristics of the elements exposed, the case study is representative of high frequency events with low to moderate consequences, along transportation corridors.

3.1. INTRODUCTION

Mountainous regions are known to be highly susceptible to rock falls events (Gardner 1970, Whalley 1984, Spang and Rautenstrauch 1988, Dorren, 2003), the Canadian Cordillera being no exception (Gardner 1977, Hungr and Evans 1989, Evans and Hungr 1993). Hence, it is not uncommon for transportation corridors through this type of terrain to be highly exposed to rock fall hazards (Brawner 1978, Pierson 1992, Bunce *et al*. 1997, Budetta 2004, Lan *et al*. 2010).

The valley formed by the Fraser River hosts an important transportation corridor between the City of Vancouver and the interior of British Columbia, and other provinces in Western Canada. This corridor cuts through the Canadian Cordillera and is used by CP, the Canadian National Railway Company (CN), and one of Western Canada's major highways (Highway 1).

The section between miles 0 and 40 of CP's Cascade subdivision (along the Fraser River west bank – Figure 3-1) has a long history of instability (Piteau 1977, Lan *et al*. 2007, Macciotta *et al*. 2011). In particular, miles 2 through 15 account for 67% of all recorded slope failure events in a length equivalent to 32.5% of the 40 miles (Figure 3-2). Instabilities documented along this section include rock, soil and snow falls, where rock falls account for more than 80% of the records. Figure 3-1(a) shows a typical slope cut along CP's Cascade subdivision. The steepness of the section of the Fraser River canyon between Boston Bar and Yale required steep slope cuts through tectonically altered rock in order to accommodate the track construction. Figure 3-1 also shows the scar and deposits of a previous event (a), the hazards of rock blocks coming to rest along the track (b) and potential future events (c) along this particular section.

Recognizing the risks associated with slope cuts in the Cordillera, CP engaged in the development and implementation of a rock slope management program (Brawner and Wyllie 1975). This system has evolved into a qualitative rating system to describe the estimated hazard and its likelihood of failure

Part of this chapter was published in: Macciotta, R., Martin, C.D., Morgenstern, N.R. 2010. Risk management of large rock slopes - state of practice. Proceedings of the 63rd Canadian Geotechnical Conference, Calgary, AB, Canada. pp. 891-898.

(Mackay 1997). Mitigation works (protection walls, ditch widening and maintenance, face stabilization and scaling) follow site inspections in the priority indicated by the rating system. This form of risk assessment and management is currently being applied at CP's Cascade subdivision.

A QRA associated with rock fall events between miles 2 and 15 of CP's Cascade subdivision was developed for this study. The assessment focuses on the risk to life of running trade employees working along this section. CP's extensive event records dating back to the 1940's served as main input for the hazard analysis, and were of significant value in the consequence analysis stage. The consequence analysis details the steps followed to estimate the freight train derailment probabilities under the scenarios considered.

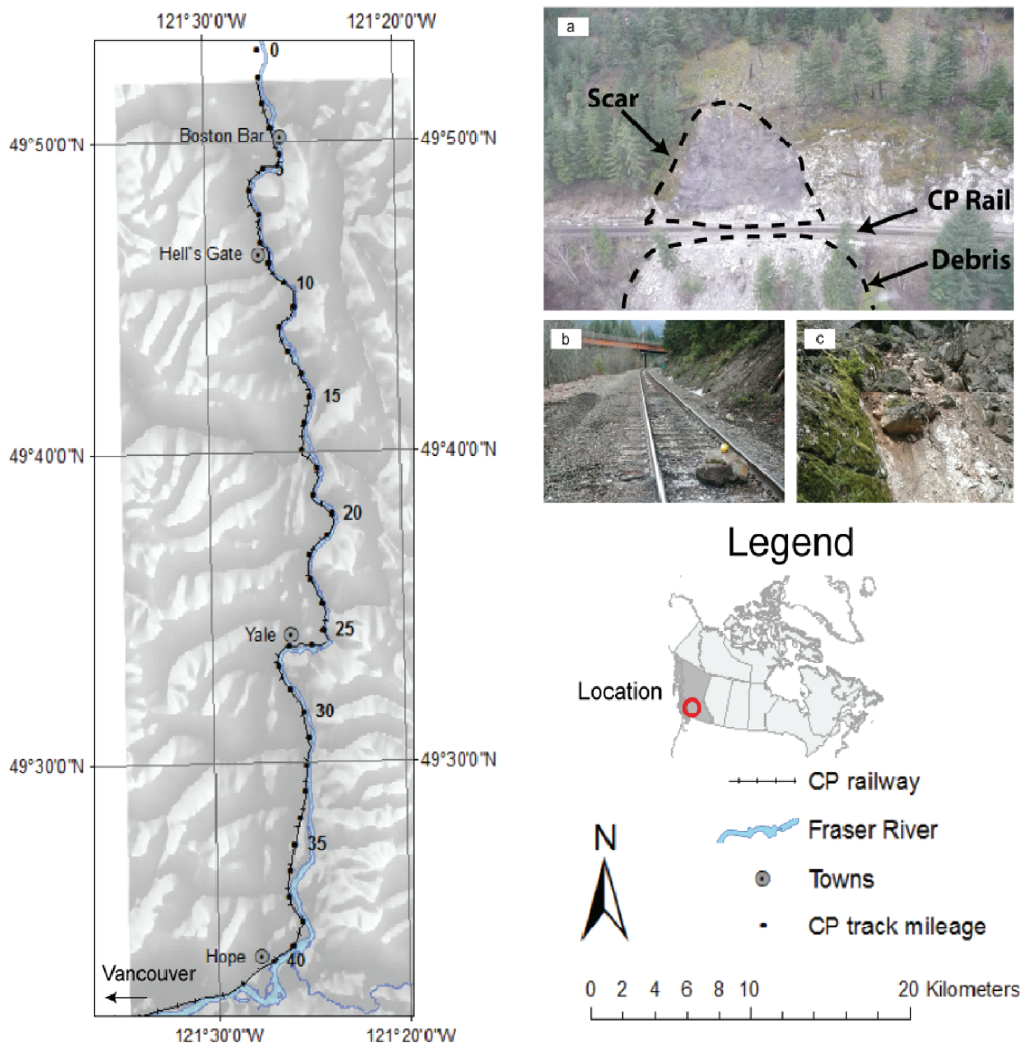


Figure 3-1CP's Cascade subdivision study area. (a) steep cut along mile 2.9, typical of the section between Boston Bar and Yale showing a scar covered with shotcrete and debris from previous events. (b) rock fall encountered during field assessment along mile 14.6. (c) unstable blocks along mile 5.7. (CP personal communication).

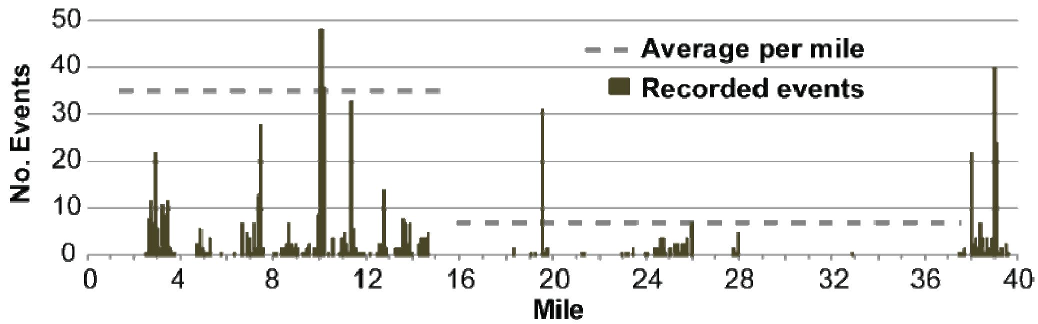


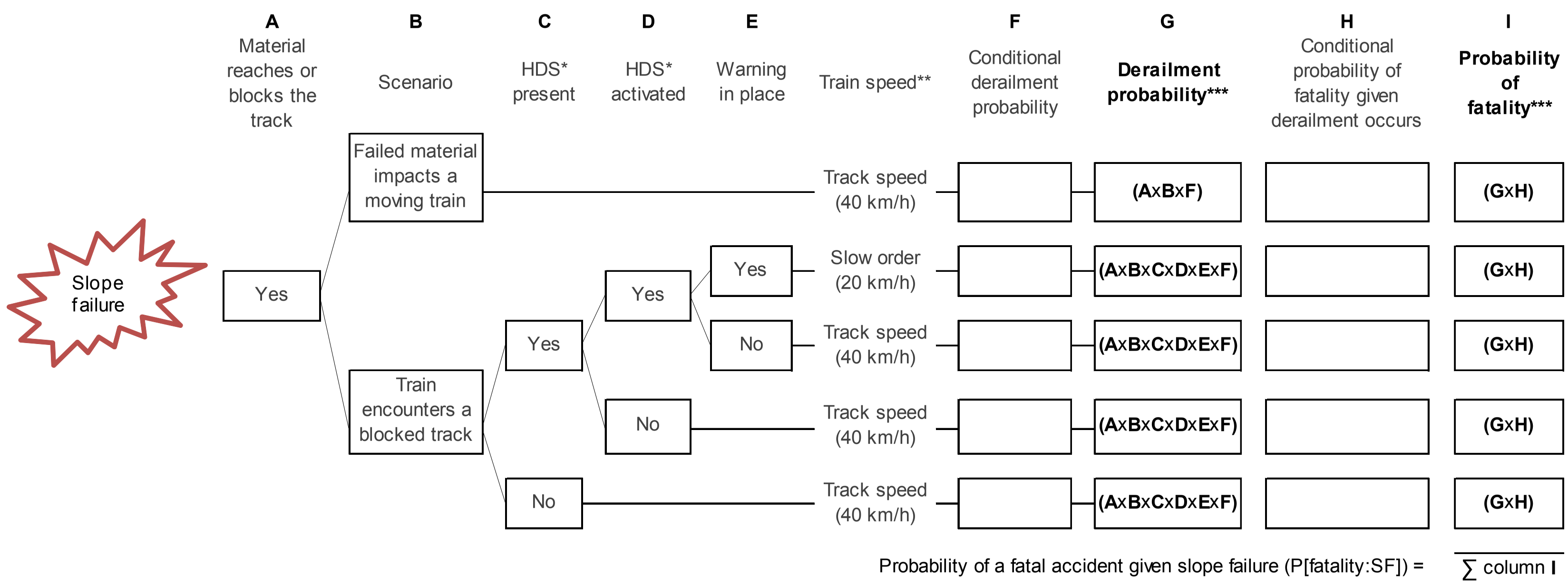
Figure 3-2 Recorded rock fall events between miles 0 and 40 of CP's Cascade subdivision.

The general methodology followed is consistent with current practice for landslide QRA (IUGS 1997, Ho *et al*. 2000, Crozier and Glade 2005, Lee and Jones 2004, Fell *et al*. 2005, AGS 2007). The estimation of the risk to life for the crew members is embedded at the end of the consequence analysis.

3.2. QRA METHODOLOGY

The process leading to a fatal accident was modelled with the aid of an Event Tree Analysis (ETA) as shown in Figure 3-3. The ETA considers two scenarios for a moving train: (1) Failed rock slope impacting a moving train, and (2) the moving train encounters a blocked track. The scenario where failed material impacts a stationary train was not considered representative of the section being analyzed. Records indicate that fatal accidents have occurred only after the train derails (CP personal communication), it was then decided to simplify the analysis by considering only fatal accidents given a derailment occurs.

Previous studies have applied QRA methods along transportation corridors (Wyllie *et al*. 1980, Bunce *et al*. 1997, Guzzetti *et al*. 2004, Pine and Roberds 2005, Shamekhi and Tannant 2010). In these analyses, multiple variables influence the location, volume, and frequency of the hazards, as well as the likelihood and severity of the consequences. This is reflected in lack of statistically valid data to stochastically derive the consequence probabilities required for QRA (derailment probability and vulnerability). When such statistics are not available, there is a need for subjective probabilities to be used. Subjective probability can be defined as an expression of personal belief about outcomes. It is a quantified measure of the degree of belief or confidence in the outcome, according to the personal state of knowledge at the time of assessment (Vick 2002). Such personal assessments are not unique and change with increased knowledge of the situation. As a consequence, when subjective probabilities are used as input for QRA, the uncertainty related to these input probabilities is then carried forward in the analysis, without proper quantification (Figure 3-4a).



Columns **A** through **F** and column **H** are input probabilities. Columns **G** and **I** are output

* HDS = Hazard Detection System.

** Train speeds are assumed given the preceding events of each branch

*** Probabilities are calculated by multiplication of the columns indicated in parenthesis

Figure 3-3 Event Tree used to estimate the probability of a fatal accident given a rock slope failure occurs - CP's Cascade subdivision study.

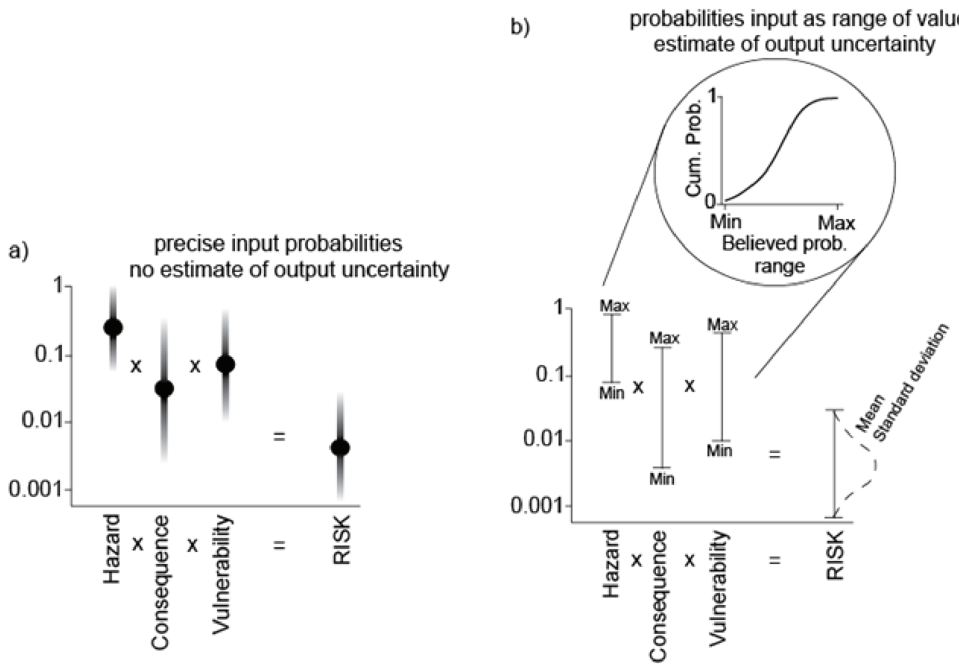


Figure 3-4 Uncertainty associated with risk analyses input variables and risk estimations. a) input probabilities as a single value with no estimate of the model output uncertainty. b) subjective probabilities defined as a range of values and a PDF.

Even when CP maintains a comprehensive record database of the study area, there are few events where freight trains interacted with the failed slopes such that the effects were noticeable. Unless extensive damage or derailment occurs, these interactions are noticed at the following inspection or are not noticed at all. Also, train characteristics such as length, speed and weight, as well as the response of the crew, will all influence the outcome when a freight train meets a blocked track. These characteristics make it difficult to develop proper stochastic approaches to populate the model processes leading to a loss. Subjective probabilities needed to be introduced in order to develop the QRA for this study.

Two analyses were developed. A first analysis uses point estimates of these subjective probabilities (a believed representative value is elicited). Given the wide range of potential slope failure volumes (rock falls less than 0.1 m^3 to rock slope failures over 5000 m^3), subjective probabilities were defined as volume-dependent. As an example, Figure 3-5 shows the defined relation between the slope failure volume and the subjective probability for a derailment to occur given a moving train is impacted by falling material. The ETA is then populated for each slope failure volume (in discrete volume increments of 0.01 m^3) to estimate the probability of a fatal accident. The total annual probability of fatality (or the annual probability of a fatal accident) is then estimated by addition over the volume range. The calculations were performed using the software Mathematica 8.0 (Wolfram Research 2010) and showed to be computationally effortless (less than one minute for the calculation). This first analysis provides a baseline risk estimate following the QRA state of practice but with no quantification of the uncertainty in the analysis results.

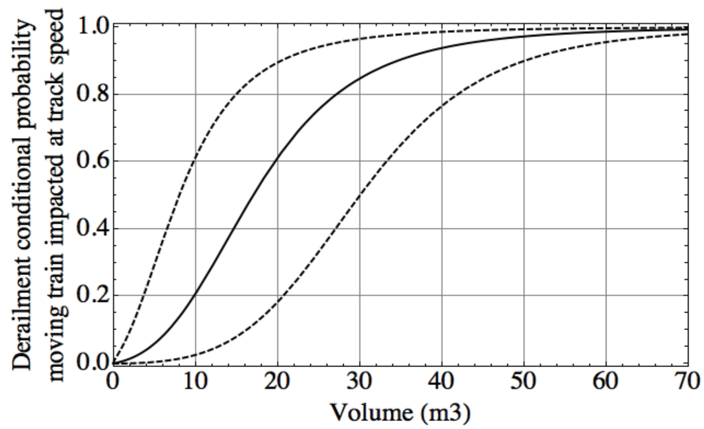


Figure 3-5 Adopted relation between the slope failure volume and the subjective probability for a derailment to occur given a moving train is impacted by falling material.

Upper and lower bounds were assigned on the second analysis for the relation between the subjective probability value and the slope failure volume (see Figure 3-5 for an example). A PDF is defined between the upper and lower bounds. This means that a PDF is generated for each volume, depending on the elicited subjective probability range. Then, a Monte Carlo simulation routine is applied. Figure 3-6 shows a flow diagram of one iteration of the Monte Carlo simulation process to estimate the probability of a fatality as per the second analysis. The simulation randomly selects a volume value for each iteration of the simulation according to the volume input PDF. Based on the selected volume, the routine then selects the subjective probability value for all input variables according to the PDFs defined for the specific volume. The outcome of the entire simulation is a PDF of the estimated probability of fatality, given a slope failure occurs. The annual probability of fatality is then calculated as:

$$P[\text{fatality}] = 1 - (1 - P[\text{fatality:SF}])^{N}$$

where:

P[fatality] = the annual probability of fatality,

P[fatality:SF] = the probability of a fatal accident given a slope failure (outcome of the event tree for each iteration of the Monte Carlo simulation), and;

N = the number of slope failures each year.

The risk to life (life loss probability) for the average crew member is then calculated as:

$$R = (P[\text{fatality}] \times DR \times C) / E$$

where:

R = the risk to life for the average crew member,

P[fatality] = the annual probability of fatality, DR = the ratio of crew members killed given a fatal derailment occurs. Estimated as 0.87 based on records of fatal derailments (CP personal communication),

C = the number of crew members per freight train, taken as two, and;

E = the total number of people employed as freight train crew that travels through the section. E was estimated at 500 employees.

E is also reduced to 100 employees in the first analysis to evaluate the effect of increasing the average worker exposure to the site. The advantages of obtaining a PDF for the estimated risks are illustrated when analyzing the results. The case study also illustrates the application of few statistical data at one of the event tree final levels for model validation. The estimated risk is then evaluated against adopted risk tolerance criteria.

3.3. POPULATION OF THE EVENT TREES

3.3.1. Hazard Analysis

CP maintains an extensive record of slope instabilities, which dates back to the 1940's. However, analysis of the relationship between rock fall volumes and their frequencies requires the use of a statistically valid recording period. According to Hungr *et al*. (1999), there are three reasons for data censoring to occur: underreported or incomplete data, too short a data interval not representative of low frequency events, and a systematic censoring resulting from the particular site conditions (the presence of protective barriers, ditches, frequency of scaling works). CP records include information on where the fallen material is encountered. It is then possible to estimate the effect of protective works by assessing the number of events retained by them as opposed to events found blocking the tracks. Systematic censoring still occurs in relation to scaling works, which modifies the volume-frequency relationship. This volume-frequency relationship reflects a system, Slope-Rail-Management process rather than the rock fall process alone.

Underreporting is a consequence of variations in record keeping standards and unnoticed events. Changes in recording standards can be assessed through slope failure histograms. Unnoticed failure events, while unavoidable, likely involve small volumes. Incomplete data in the records analyzed for this study was also found in the form of missing volume information. Slope failure histograms were analyzed to assess the changes in record keeping standards. The data set was grouped in volume ranges showing the same trends in time. Figure 3-7 presents the number of recorded failure events per year for the grouped volume ranges. recorded failures with unknown volumes seemed to correspond to volumes of less than 1 m^3, which is consistent with the findings of Hungr *et al*. (1999) when analyzing railway and highway rock fall event data in the area. Failures with volumes over 1000 m^3 were excluded due to their low frequency when compared to rock falls of lesser volumes. No censoring is assumed to occur for these larger failures given their size and consequences in this particular section of the railway.

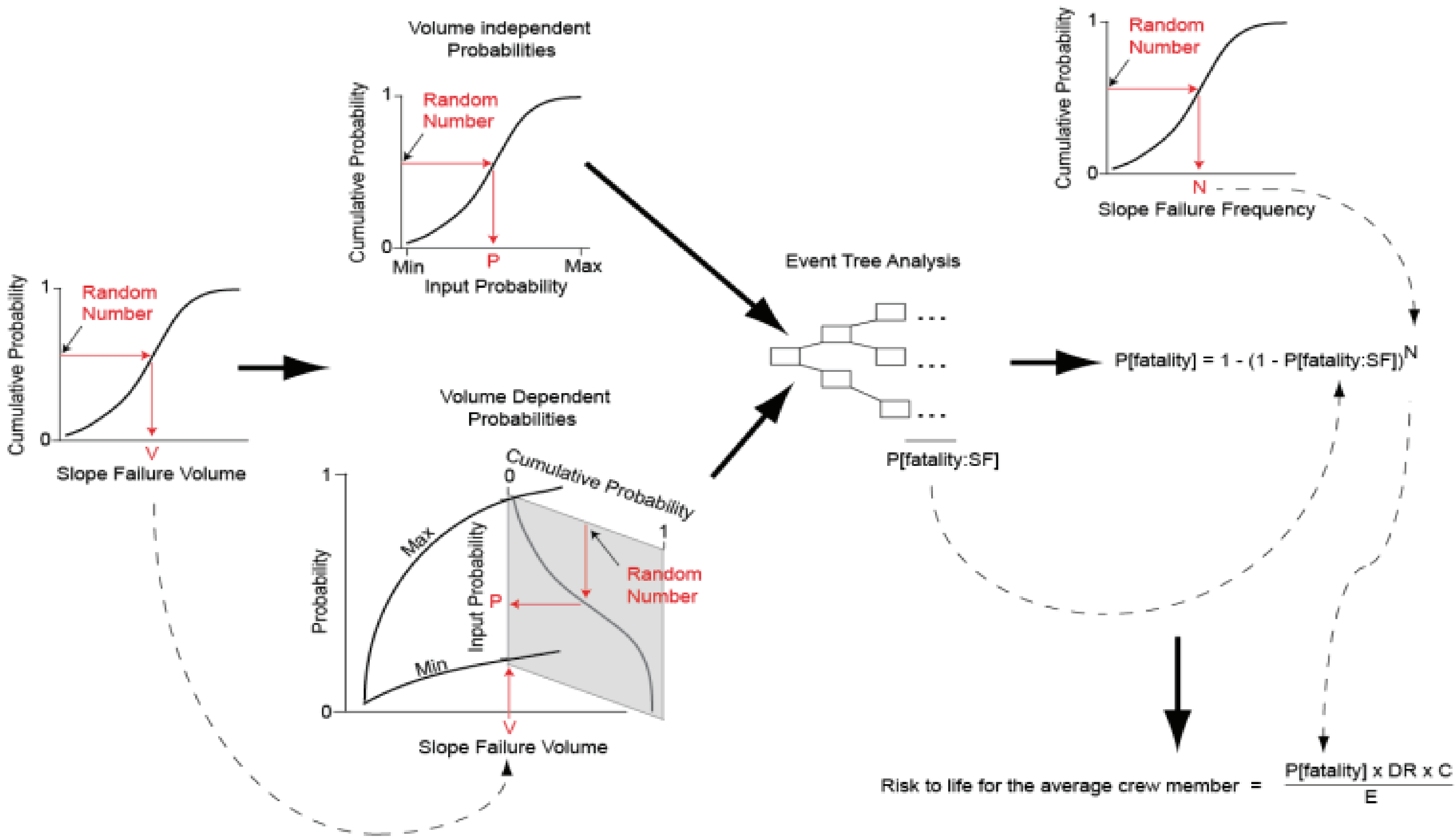


Figure 3-6 diagram of one iteration of the Monte Carlo simulation process to estimate the probability of a fatality - Cp's Cascade subdivision study.

Records used for the censoring analysis correspond to miles 0 to 40 of CP's Cascade subdivision. This assumes that even when the recording standards varied with time, they remained spatially consistent along this section. This data set corresponding to miles 0 to 40 was used to ensure enough records to confidently define the statistically valid time intervals. The data corresponding to miles 2 through 15 are then used to estimate the frequency of slope failures along these 14 miles.

Hungr *et al*. (1999) suggested that a strong indication of censoring is a sharp increase in apparent frequency with time. They defined the earliest year of each valid recording interval based on the slope of a best-fit linear regression for the histograms. Assuming the latest year of our valid interval to be the last year of complete records (in our case 2009), the slope of the linear regressions can be calculated for different starting years. All slopes are initially positive, which indicates skewness to the right, and eventually approach to zero (horizontal line). This is taken as an indication that the data within that interval are consistently being reported. The slopes will fluctuate further between positive and negative values responding to the natural variation of the data. Figure 3-8 presents plots of the histogram linear regression slopes against the first year of the corresponding time interval. All time intervals include up to 2009.

The start of each valid recording period was defined based on the linear regression slopes, and trying to maximize the number of records for statistical validity. The time intervals considered for the subsequent frequency analysis are presented in Table 3-1.

Event volume range (m^3)	Interval considered
Up to 100	1975 – 2009
100 to 1000	1987 – 2009

Table 3-1 Time intervals of statistically valid records considered for the failure frequency estimation - CP's Cascade subdivision study.

Failure events with volumes between 100 and 1000 m^3 have a shorter time interval when compared to smaller events. It is expected larger events would be consistently recorded at earlier stages than smaller events, however our data set indicate otherwise. It is likely that large events occurred at early stages but, due to underreporting, no volume estimations were noted and the events are hidden within those with no specified volume.

3.3.2. Slope Failure Annual Frequency, Volume Distribution and Volume - Cumulative Frequency Curves

Slope failure volumes in the study area cover several orders of magnitude. As such, a unique event frequency is not enough to characterize the hazard. A Volume-Cumulative Frequency curve (VCF) was developed for the events reported between miles 2 and 15 within the statistically valid time intervals shown in Table 3-1. The VCF is presented in Figure 3-9. A power law regression was considered appropriate for volumes above 0.6 m^3. Extrapolation up to 1000 m^3

seemed reasonable and conservative. The cumulative frequency tends to flatten towards the horizontal at smaller volumes, most likely due to a combination of underreporting and the real physics of the phenomenon. A linear regression analysis was carried out for the volumes below 0.6 m^3. Note this has the implication that events with volumes between 0.01 and 0.6 m^3 have the same non-cumulative frequency (slope of the linear fit).

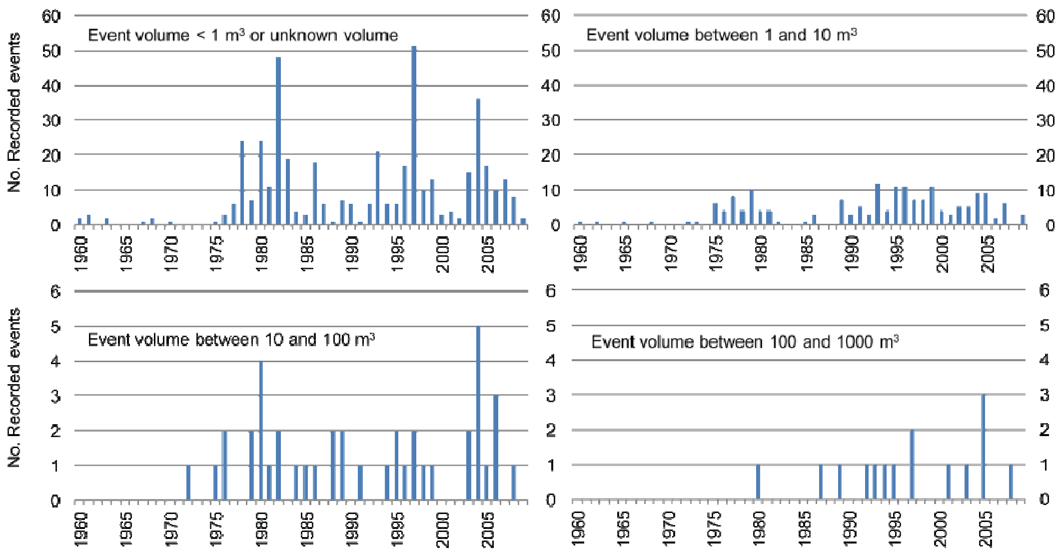


Figure 3-7 Recorded failure events along miles 0 to 40 of CP's Cascade subdivision plotted against year of occurrence.

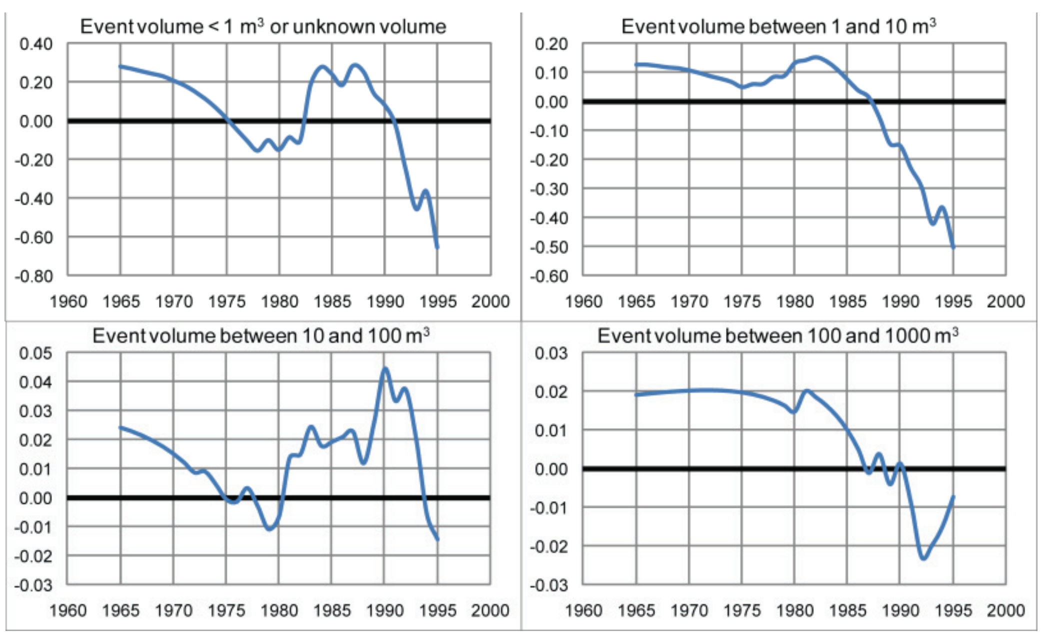


Figure 3-8 Slope of the linear regression for the recorded event histograms along miles 0 to 40 of CP's Cascade subdivision. The vertical axe is the linear regression slope. The horizontal axe is the first year of the assumed time interval (data up to 2009 inclusive).

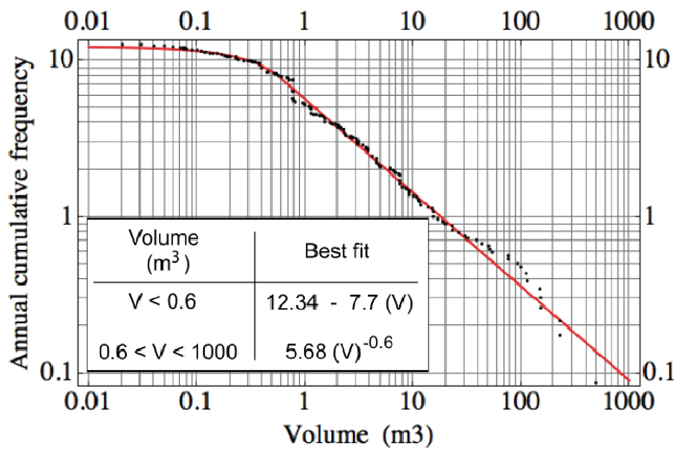


Figure 3-9 Volume-Cumulative frequencies estimated from records between miles 2 and 15 of CP's Cascade subdivision.

The VCF was used to obtain cumulative probability distributions of the annual number of slope failures and their volumes. These distributions are presented in Figure 3-10 and Figure 3-11. The records of annual slope failures were approximated with an Inverse Gaussian distribution and the volumes were approximated with a Pearson distribution.

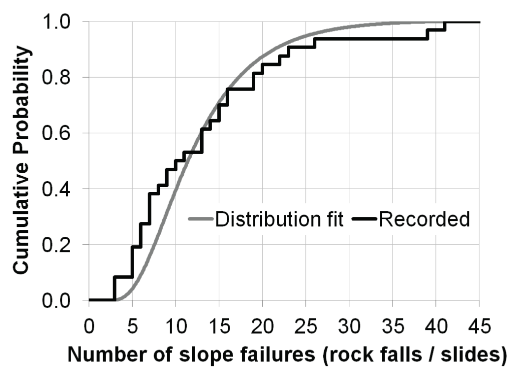


Figure 3-10 Cumulative probability distribution of the annual number of slope failures (rock falls and slides) - CP's Cascade subdivision study

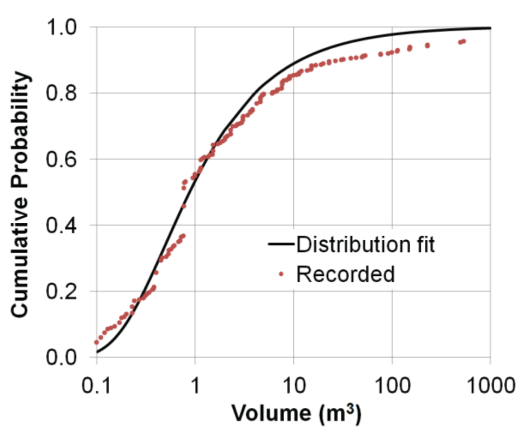


Figure 3-11 Cumulative probability distribution of slope failure volumes - CP's Cascade subdivision study.

3.3.3. Probability the Failed Material Reaches / Blocks the Track

CP's records include failures where the material was encountered blocking the tracks and those caught within a ditch or behind protective structures. The fraction of failures that reach the track and can potentially block it or impact a moving train needs to be estimated. The ratio between records where the material was encountered blocking the tracks to total number of slope failures was calculated. This calculation was done for three volume ranges in order to assess any volume dependency. Increasing the number of volume ranges limited the number of records within each range to a level where ratios calculated became unreliable. Table 3-2 presents the calculated ratios.

Event volume	Total No. events	No. events track blocked	Track blocked ratio
All events	535	156	0.29
0.1 – 1 (m^3)	153	40	0.26
1 – 10 (m^3)	135	52	0.39
10 – 1000 (m^3)	40	21	0.53

Table 3-2 Ratio of failure events blocking the track to total failures - CP's Cascade subdivision study.

The data presented in Table 3-2 correspond to failures recorded between miles 0 and 40 of CP’s Cascade subdivision. It can be argued that other sections different from miles 2 through 15 can show different ratio of events blocking the track to total number of events. However, the fact that 67% of failures are reported between miles 2 and 15 and the need to maximize the data for statistical purposes, lead to the use of the more extensive record. Discretization into smaller volume ranges will not increase the accuracy of the estimations given the record extent. The results in Table 3-2 illustrate that the probability of a block reaching the track increases with increasing volume. Hence a continuous probability distribution can be defined. Knowing that some failures that blocked the track may not have been reported, a conservative approach for the continuous probability distribution was selected for the initial analysis (Figure 3-12).

There is not enough information on volumes less than 0.1 m^3 to confidently extrapolate the linear assumption shown in Figure 3-12. Consequently the probability distribution was truncated to a minimum of 0.3 for these smaller volumes. Also, only a few failure volumes larger than 1000 m^3 have taken place and all resulted in blocking the tracks. It was therefore assumed that any failure over 1000 m^3 would certainly block the track.

The estimated ratio directly represents the probability for a slope failure to block the track, without impacting any train while coming down the slope. Also, any block rolling past the track and into the river will not be accounted for. These ratios were considered crude approximations of the probability that the material reaches the track or blocks the track, depending on the scenario being analysed.

As such, for the second QRA subjective probability maximum and minimum values were adopted (Figure 3-13).

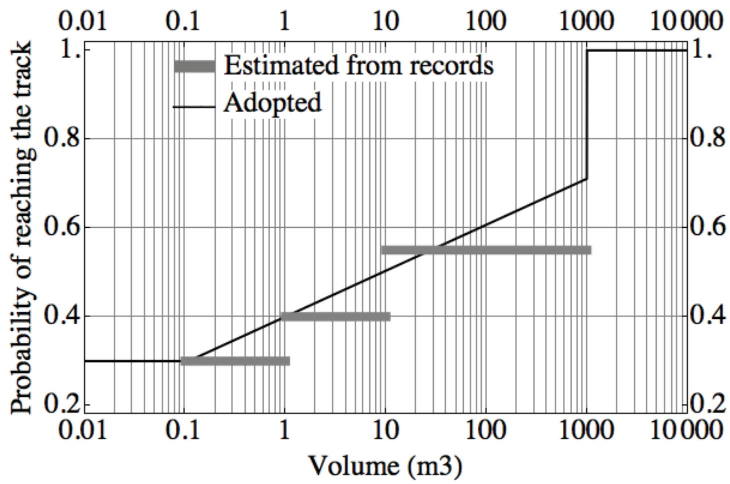


Figure 3-12 Derived conditional probability distribution for an event reaching the track given a slope failure occurs for the initial QRA - CP's Cascade subdivision study.

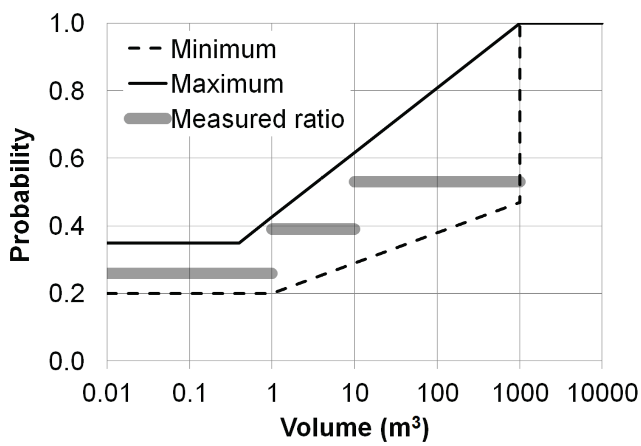


Figure 3-13 Derived conditional probability distribution for an event reaching the track given a slope failure occurs for the second QRA - CP's Cascade subdivision study.

3.3.4. Probability that a Failing Slope Impacts a Moving Train.

To simplify the analysis, two outcomes were considered after a slope failure reaches the track: the material impacts a train, or the material blocks the track. these then become mutually exclusive events, and their probability, given the material reaches the track, sum one.

The impact probability was estimated using the Binomial Theorem (Bunce *et al*. 1997) for the initial QRA. This calculation gives the probability of one or more impacts between a train and a failure of volume vi:

$$P[I]_{vi} = 1 - (1-P[S])^{f(vi)}$$

where:

P[S] = spatial probability of failing slope coinciding with moving train, and;

f(vi) = annual frequency of the failure of volume vi (obtained from the VCF curve) multiplied times the probability the material reaches the track.

The spatial probability P[S] is estimated as:

$$P[S] = (L \times T) / (V \times 24)$$

where:

L = average train length in Km,

T = average number of trains per day, and;

V = train speed in Km/h.

Each iteration of the Monte Carlo simulation developed for the second QRA is independent, and the impact probability is calculated as the probability that the material reaches the track multiplied times P[S]. Random sampling of the slope failure volume, which depends on its cumulative probability distribution (Figure 3-11), and use of the Binomial Theorem to account for the slope failure frequency, accounts for the failure volume-dependent frequency. Table 3-3 presents the input parameters used to calculate P[S] for both QRAs and the estimated average, upper and lower bounds.

	Initial QRA	Second QRA	
		Lower bound	Upper bound
L	2 km	1 km	3 km
T	20 trains/day	20 trains/day	25 trains/day
V	40 km/h	40 km/h (posted track speed)	
P[S]	0.042	0.021	0.078

Table 3-3 Input parameters used to estimate the spatial probability of a failing slope coinciding with a moving train and estimated average, upper and lower bounds for P[S] - CP's Cascade subdivision study.

A uniform probability distribution was assumed for the values of L and T (discrete distributions) within the limits given above for the second QRA. Note that this calculation considers constant train frequency and even slope failure probability throughout the day and throughout the year. It can be argued this is not representative of the site, however, this simplification was considered a valid approximation for the evaluation of annual average risks. Posted track speed of 40 km/h assume no previous slope instabilities reported at the time the train travels through the section and no slow orders to be in place.

3.3.5. Probability that a Freight Train Encounters a Blocked Track

The probability of a freight train encountering a blocked track depends on the probability of the failed material reaching – and blocking – the track and the probability that it does not impact a moving train when falling. This was estimated as (1 - P[I]) for the initial QRA and as (1 - P[S]) for each iteration of the simulation in the second QRA.

There is the possibility that the first vehicle reaching the blocked track is not the vehicle-type of the analysis. The frequency of failed material blocking the track needs to be combined with the probability that this vehicle type is the first to reach the blocked track. A simple ratio of the number of vehicles types in the analysis to the total number of vehicles using the track can be used:

P[vehicle type i] = (No. vehicles type i) / (total No. of vehicles)

This study is concerned with the derailment probability of freight trains and the risk to life for their crew. Because the other vehicle types have a low frequency along this track section, it was assumed that a freight train vehicle will be the first to encounter a blocked track. Given that this vehicle type is the most frequent, this assumption was not considered overly conservative.

3.3.6. Probability the HDS is Present and Running

The Hazard Detection System (HDS) consists of a series of wire fences along the section, between the railway track and the cut slope. The spacing between wires is about 20 to 25 cm and the fence height varies between less than a metre and up to 2 m in some sections. When a section of track is blocked, it is expected that the material blocking the track would have broken one or more of these wires in its path. This is detected by the system and the nearest track circuit signal shows a track occupation.

The probability that the HDS is present depends on the ratio of section length where the HDS is installed to the total section length. This probability also depends on the total number of days per year the system is active. The HDS is installed along the entire mileage of the study section. Considering the time required for maintenance and repairs, the probability that the HDS is present was consider to range between 0.9 and 1 for any given year. A uniform PDF was used between this range as input for the second QRA. For the initial QRA, an estimate of 0.98 was used, based on the system being down for 1 week each year.

3.3.7. Probability the HDS is Activated

The probability that the HDS is activated, given the track gets blocked, depends on the failure volume, the spacing between wires and the position of the system with respect to the slope. Figure 3-14 highlights the HDS setup with respect to the slope at different miles within the study section.

A number of CP's records include information on the HDS activation. However there are insufficient records that also include information on where the material was encountered (blocking the track or not) and the failure volume for a statistical estimation of the HDS activation probability for different block volumes. Based on the characteristics of the HDS and the slopes along the study area, judgement was used to estimate the HDS activation probabilities. These adopted probabilities and their justification are presented in Table 3-4.

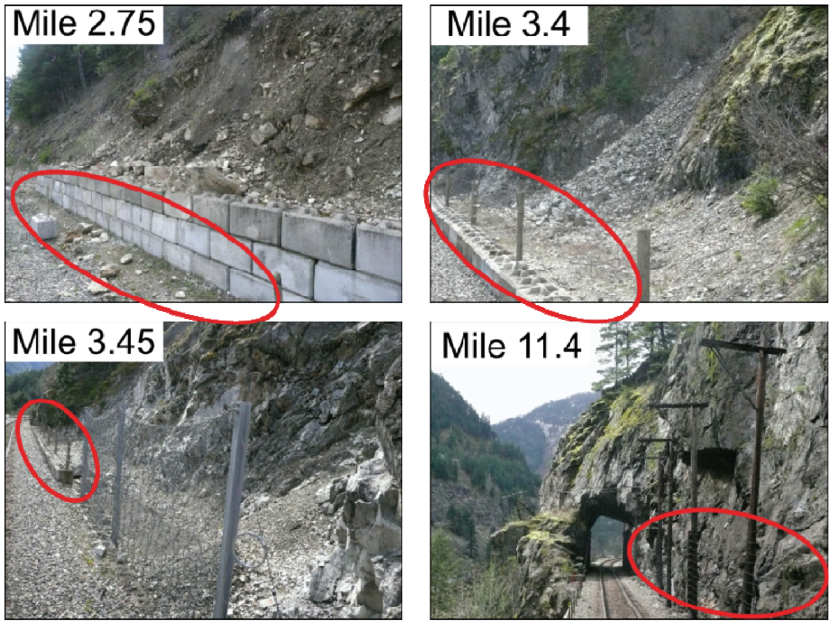


Figure 3-14 HDS setup at different locations - CP's Cascade subdivision study.

Volume (m^3)	Assumed probability HDS is activated		Justification
	Initial QRA	Second QRA	
0.01	0.01 (residual)	0.01 to 0.05 (residual)	A 10 to 20 cm diameter block can easily jump the wire fence or pass between wires.
0.1	0.5 (even chance)	0.1 to 0.5	A 30 to 50 cm diameter block activates the wire fence when rolling through it, but can easily jump over the fence depending on the slope section.
10	0.85	0.6 to 0.9	A 1.5 to 2 m diameter block activates the wire fence when rolling through it, but may jump over the fence depending on the slope section.
100	1 (certain)	1 (certain)	A 4 m diameter block breaks the fence and reaches the track as a pile of debris. This activates the HDS

Table 3-4 Subjective probabilities for the HDS activation conditional probability - CP's Cascade subdivision study.

Based in Table 3-4, continuous probability distributions are defined for the initial and second QRAs as shown in Figure 3-15 and Figure 3-16 respectively.

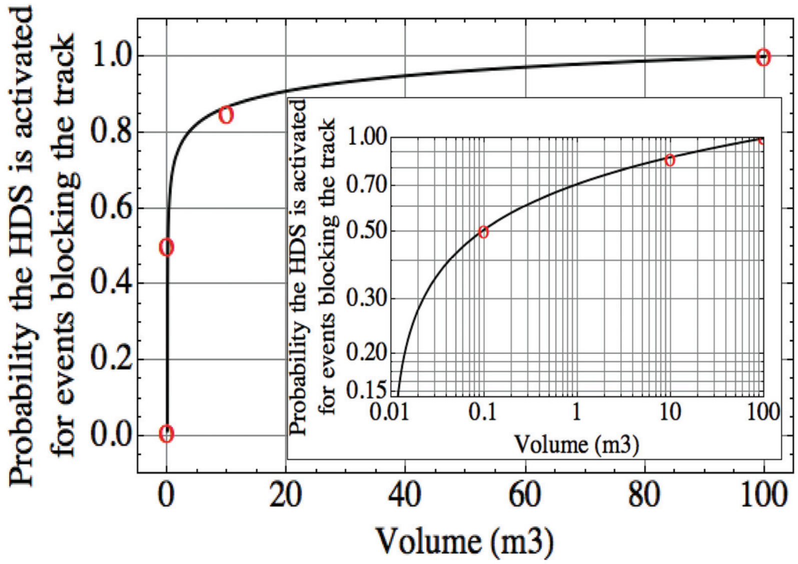


Figure 3-15 HDS activation probability distribution adopted for the initial QRA - CP's Cascade subdivision study.

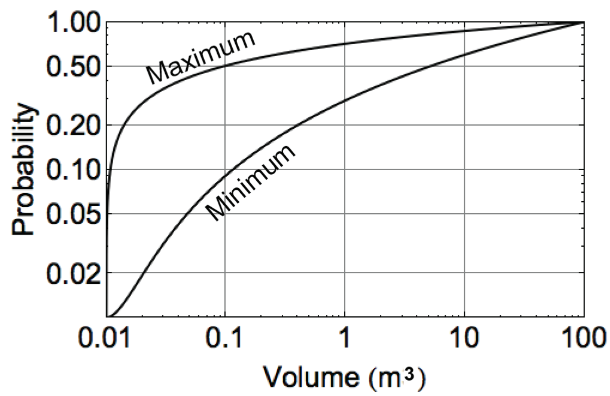


Figure 3-16 HDS activation probability distribution ranges adopted for the second QRA - CP's Cascade subdivision study.

3.3.8. Probability a Warning is Issued and Train Speeds

When a section of the HDS is activated, the nearest track circuit signal shows a track occupation and activates a slow order. This implies that the first train to encounter the blocked track receives a warning only if there is a track circuit signal between the activated HDS and the train. The section between track circuit signals is known as the signal block. It is assumed that if the train is outside the signal block where the event occurs, and the HDS is activated, the warning is effective.

The probability that a warning is issued can then be assumed to be the complement of the probability that the train is inside the signal block when the event occurs:

$$P[warning] = 1 - P[SignalBlock] = 1 - (B \times T) / (V \times 24)$$

where:

P[warning] = probability a warning is issued given the HDS is activated,

P[SignalBlock] = probability the train is inside the signal block when the event occurs,

B = distance between the activated HDS and the nearest track-side signal. As this is not known, it can be conservatively estimated as the entire length of the signal block, or as half this length to account for an average distance. In the study area this distance is assumed between 0.5 and 1km,

T = number of trains per day, and;

V = train speed in Km/h.

Table 3-5 presents the input parameters used to calculate P[warning] for both QRAs and the estimated average, upper and lower bounds. Uniform probability distributions were adopted for the values of B and T within the limits given for the second QRA. It can be seen that the input values characteristic of the site render a high warning probability given the HDS is activated and show limited variation.

	Initial QRA	Second QRA	
		Lower bound	Upper bound
B	1 km	0.5 km	1 km
T	20 trains/day	20 trains/day	25 trains/day
V	40 km/h	40 km/h (posted track speed)	
P[warning]	0.98	0.99	0.97

Table 3-5 Input parameters used to estimate the probability a warning being issued given the HDS is activated - CP's Cascade subdivision study.

The analyses assume that if warned, the train is travelling at restricted speed. This restricted speed, or slow order, is taken, as a maximum, to be half the track speed (20 km/h). Records show that most slow orders in this section are to keep speeds of about 16 km/h, which is consistent with the above assumption. All other branches with unsuccessful warning outcomes consider the train to be travelling at track speed when encountering the blocked track.

3.3.9. Conditional Derailment Probability - Falling Material Impacts a Moving Train

When the falling material impacts a moving train, the derailment conditional probability given an impact occurs is a function of the material kinetic energy. This energy depends on the failed mass (or volume), velocity, and how the block disaggregates and reaches the track. A comprehensive analysis of these factors is a complex matter requiring information rarely available. Furthermore, information on trains being hit by falling blocks can be unreliable. Unless the impact caused a derailment or excessive damage, it is only noticed when the train reaches the next inspection site. In a simplified and practical approach, the derailment conditional probability given the failing material impacts a moving train was defined as a function of the failed volume, where upper and lower limits for the subjective probabilities were elicited based on the limited available data. According to Bunce (2008) there hasn't been a rock fall of less than 1 m^3 reported by CP that caused a derailment after impacting a moving train. It was considered the derailment conditional probability to be negligible for these slope failure volumes. This probability should then increase with increasing volume. It was estimated that for volumes approaching 40 to 50 m^3, the derailment probability given impact occurs should approach certainty. A wide range of subjective probabilities was then adopted for slope failure volumes between 1 and 40 m^3, reflecting the uncertainty at this level of the ETA. Figure 3-17 and Figure 3-18 show the adopted derailment probabilities given the failed material impacts a moving train for the initial and second QRAs respectively.

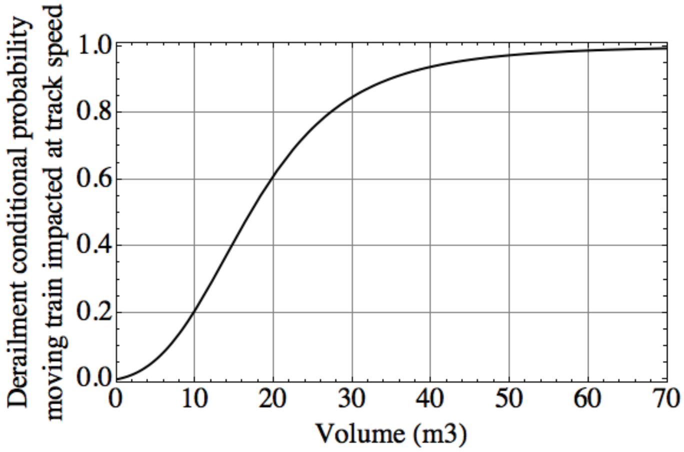


Figure 3-17 Adopted derailment conditional probability distribution given a rock fall impacts a moving train at track speed (40 km/h) for the initial QRA - CP's Cascade subdivision study.

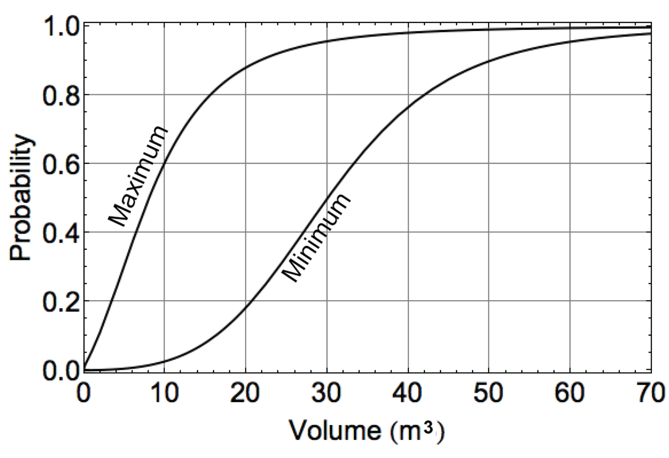


Figure 3-18 Adopted limits for the derailment conditional probability distribution given a rock fall impacts a moving train at track speed (40 km/h) for the second QRA - CP's Cascade subdivision study.

3.3.10. Conditional Derailment Probability - Train Encounters a Blocked Track

When a slow order is issued, the speed should be slow enough so that the train crew are able to stop the train should they observe the blocked track. This distance between the train and the farthest visible section of rail is referred to as the sight distance. The distance required for a train to stop is the stopping distance. The ratio of sight to stopping distance can aid in estimating the probability that the train stops before impacting a blocked track. A ratio of 1 indicates there is just enough track between the train and the blocked section for the freight train to come to a stop. Lower ratios indicate the train is not able to stop but only reduce its speed.

The stopping distance is a complex field and depends on a variety of factors such as train characteristics (length, weight, type, brake force, initial speed), alignment characteristics (grade, curvature), interaction between the wheel and track, and weather conditions, among others (Barney *et al*. 2001, Loumiet and Jungbauer 2005, Bunce 2008). Loumiet and Jungbauer (2005) presented an analysis of the stopping distance for a freight train consisting of 100 loaded cars and 4 locomotives. This represents a train of similar characteristics to the average freight train considered in this study. The relationship they found between the freight speeds and stopping distance is reproduced in Figure 3-19. To account for variations between the conditions for our study and those used by Loumiet and Jungbauer, a slightly higher stopping distance was estimated (see Figure 3-19).

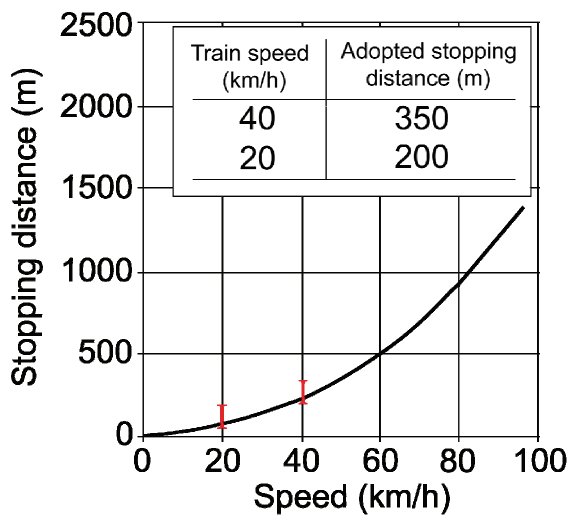


Figure 3-19 Relationship between freight speeds and stopping distance after Loumiet and Jungbauer (2005) - CP's Cascade subdivision study.

The average sight distance (visible track length ahead of the locomotive) is about 1 km, enough to stop the train given the crew members see the obstacle and react accordingly. However, a few narrow turns decrease this sight distance to about 150 to 200 m.

Judgement was required to account for the volume of material blocking the track. It was judged that up to 0.1 m^3 of material poses a residual probability of derailment (0.01). It was also judged that when 100 m^3 or more material blocks the track, chances of derailment approach certainty, given the impact is at enough speed. A range of subjective probabilities between 0.1 and 1 was adopted for material volumes of about 10 m^3. This analysis was used to define upper and lower limits for the subjective probabilities of derailment. Figure 3-20 presents the limits adopted for the subjective probability of derailment after a train reaches a blocked track and the distribution used for the initial QRA (red dashed lines). Uniform probability distributions for each volume where adopted in the second QRA between the limits shown in Figure 3-20.

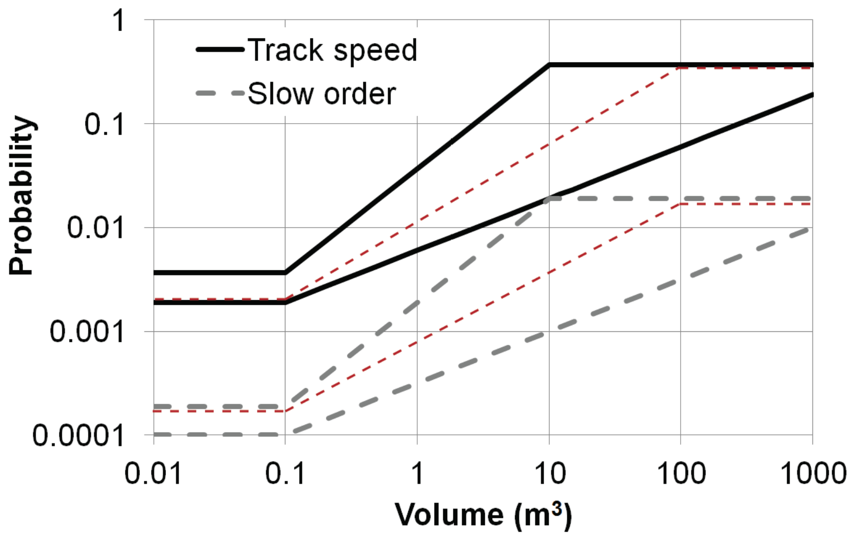


Figure 3-20 subjective probability that derailment occurs after a train reaches a blocked track - CP's Cascade subdivision study..

3.3.11. Probability of Fatality Given a Derailment Occurs

Bunce (2008) presented an analysis of CP's records showing that only 3 out of more than 230 mainline derailments resulted in fatal accidents. This suggests that in average 1.3% of all derailments results in a fatal accident. These statistics include cases in a range of conditions along CP operations. Bunce noted that all fatal derailments had occurred when the locomotive derailed and fell into a water body, and also suggested that the probability of a fatal accident given a derailment in mountainous terrain would be higher than average. However, due to the history of instability in the study area, the track speed is limited when compared to other sections along the Canadian Cordillera (40 km/h against more than 60 km/h at some sections). Table 3-6 shows the adopted values for the conditional probability of fatality used to populate the ETA. Following the approach presented in Bunce (2008), upper and lower limits corresponding to the train travelling at slow order speed were adopted as one order of magnitude lower than those for track speed.

Train speed	Initial QRA	Second QRA	
		Lower limit	Upper limit
Track speed (40 km/h)	0.013	0.002	0.05
Slow order (20 km/h)	0.0013	0.0002	0.005

Table 3-6 Adopted conditional probability of fatality given a derailment occurs - CP's Cascade subdivision study.

3.4. RESULTS

The outcome of the Monte Carlo simulation applied to the event tree is a normalized histogram of observations that can be approximated to a PDF of the estimated risk values. Point estimates of the resulting PDF (mean, mode, standard deviation) can be easily compared against selected risk evaluation criteria. Monte Carlo simulations select random values for the model parameters at each iteration. As such, point estimates derived from the resulting PDF will vary for different Monte Carlo simulations on a same model. Incrementing the number of iterations increases the number of results to a larger statistical sample, and reduces this variability. Ten simulations for each of four different number of iterations were evaluated. Results are plotted in Figure 3-21 in terms of the mean and variance of each simulation result. It was decided to work with the results obtained for a simulation with 100 000 iterations, given its variability was considered negligible (Figure 3-21).

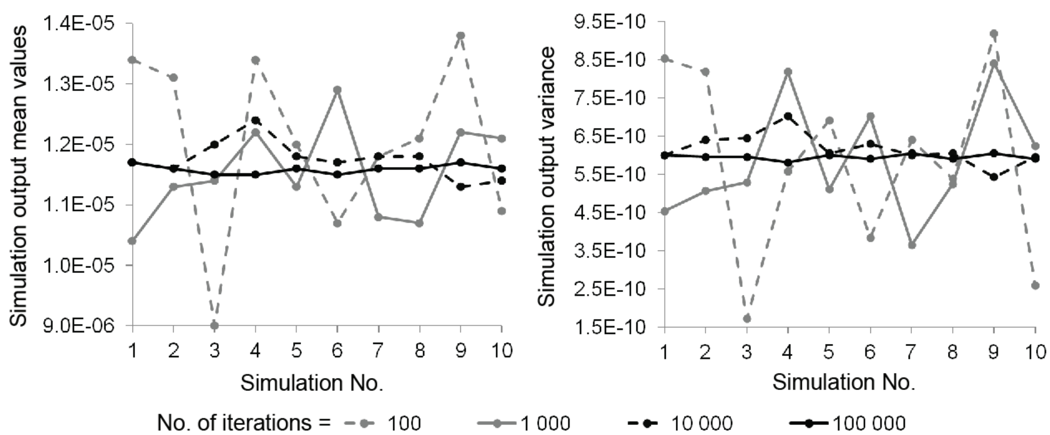


Figure 3-21 Comparison of Monte Carlo simulation outputs (mean and variance of results) for increasing number of model iterations - CP's Cascade subdivision study.

When simulation outcomes cover several orders of magnitude, the histogram of results will show a long tail towards the higher magnitudes. The mean value of results cannot be adopted as a measure of their central tendency. Calculation of the mean weights each outcome by its resulting value, thus shifting it towards higher values (Figure 3-22). Adopting the mode of the PDF is a way to assess the central tendency of the results, however the shape of the distribution towards the lower values (and in the vicinity of the mode) remains hidden by the scale of the plot.

In order to analyse the central tendency and variability of the estimated risk, the obtained PDF was plotted in semi-logarithmic scale (base-10 logarithm of the risk values). The PDF in the semi-logarithmic scale approximated a normal distribution. Point estimates (mean and standard deviation) can then be calculated using the base-10 logarithm of the model output. This method minimizes shifting of the calculated mean, while the mode suffers no change, and allows for a better assessment of the distribution.

It is important to note that this approach treats the risk orders of magnitude as risk categories, minimizing the effect of the actual estimated values when calculating the point estimates. However, it is believed the approach is compatible with how probability is perceived at orders of magnitude below 10^{-1}, and compatible with how evaluation criteria are expressed. Further, it allows for a measure of the uncertainty in the estimated risks.

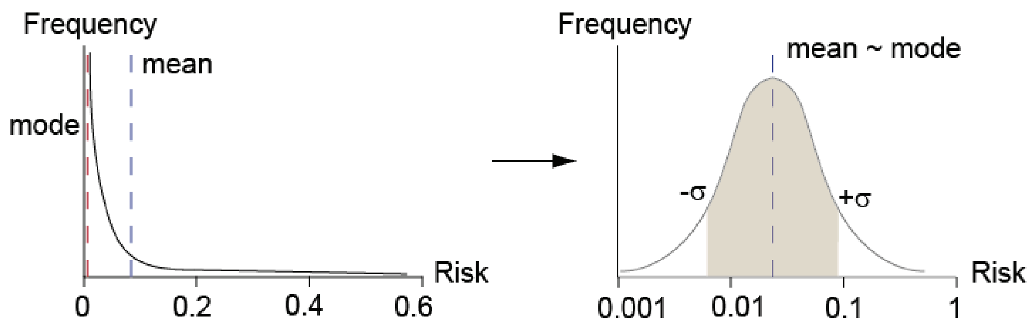


Figure 3-22 Diagram of Monte Carlo simulation results covering several orders of magnitude. Plot in normal scale (left). PDF when plotted in semi-logarithmic scale is shown on the right (assuming normality).

3.4.1. Derailment Probability

The annual probability of derailment for the initial QRA was estimated at 0.081, or a frequency of 1 derailment every 12.4 years. Records along the study area indicate one freight derailment between the years 1975 and 2009 (or a frequency of 1:35 years). However this is an average derailment frequency for the period, with fewer trains using the corridor in the early years. Assuming that between 1980-90 and 2009 the train frequencies were similar to those considered in the analysis, the derailment frequency from records increases to 1 in 20 to 30 years. Considering that during this period of time a second derailment of a maintenance vehicle occurred, the derailment probability could be taken as closer to 1 in 15 or 20 years. These derailment frequencies estimated from CP records are similar to those estimated following the procedure in the initial QRA.

Figure 3-23 shows the derailment annual probability distribution calculated in the second QRA. The derailment annual probability mean value estimated by the model was 0.04 with a mode of 0.02, consistent with the statistical data (0.03 if considering the one freight derailment in 30 years shifting towards 0.1 if considering the maintenance vehicle derailment within a period of 20 years).

Analysis of the estimated derailment probability variation is less straight forward. In probabilistic terms, having one or two observations within a period of time does not allow for a reliable approximation of event frequency. No estimation of the annual frequency variation can be obtained either. The following reasoning was followed to evaluate the model variation. The fact that there was only one freight train derailment in 20 to 30 years can lead to the belief that there should be a limited probability for the derailment frequency to be 1 in 100 years. On the other hand, given a second derailment occurred on the section (a maintenance vehicle) it is believed a limited probability would correspond to a frequency of 1 in 10 years and a residual probability to 1 in 2 years. These limits are then compared to the model's PDF for the estimated derailment probabilities (Figure 3-23). Following this methodology, the model was assessed to have enough accuracy in light of the available information for the estimation of derailment probabilities. It should be noted that this approach showed the potential to allow for model calibration as statistical data increase with time.

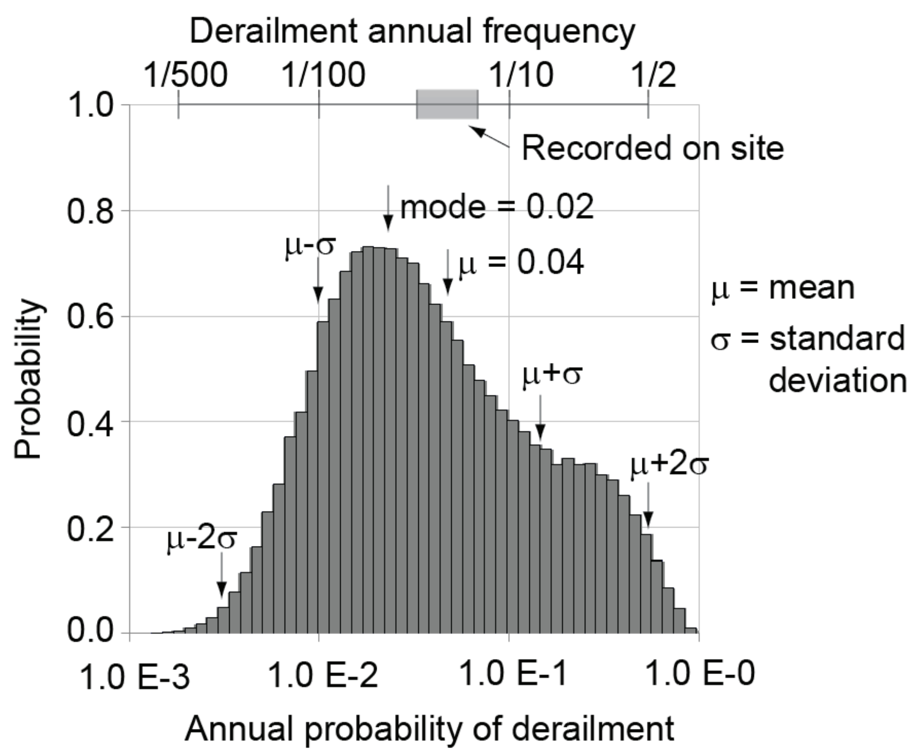


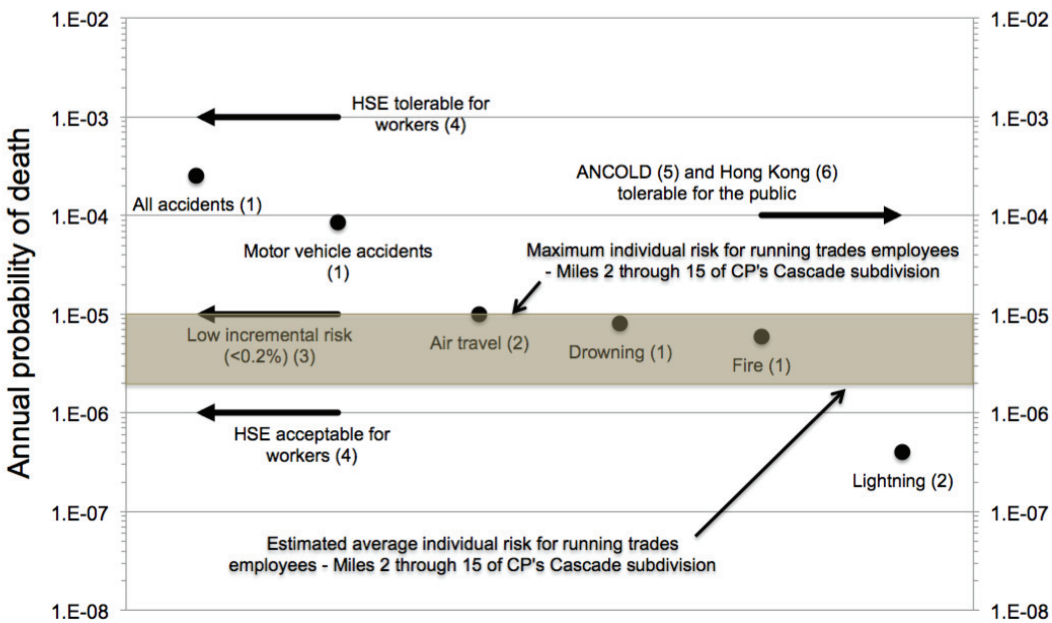
Figure 3-23 Monte Carlo simulation output for the annual derailment probability in the second QRA - CP's Cascade subdivision study.

3.4.2. Individual Risk to Life

In the initial QRA, the individual risk to life for the average crew member travelling along miles 2 through 15 of CP's Cascade subdivision was estimated at 2×10^{-6} when considering 500 crew members working along that section.

Accounting for the assumed number of trains per day (20) and considering each crew consists of 2 members, there will be about 40 running trade employees travelling through the section each day. Accounting for travelling times and that the return trip is done along CN's track, on the other side of the river, there would be a minimum of about 100 employees running trades along the study area. The maximum value of the individual risk (or risk to the crew member most exposed to the hazardous section) can then be approximated to 1 x 10^{-5}.

Figure 3-24 presents the calculated individual risk for the average crew member and the worker most exposed. Also shown are some adopted risk evaluation criteria and common risks for comparison.



(1) Derived from the 2007 age-standardized mortality rates for the Canadian population (Statistics Canada 2010).
(2) Data from Baecher and Christian (2003).
(3) Porter's suggestion that the incremental risk is low if it doesn't exceed 0.2% of the Canadian age- standardized risk of loss of life (Porter et al. 2009).
(4) HSE (2001).
(5) ANCOLD (2003).
(6) ERM (1998).

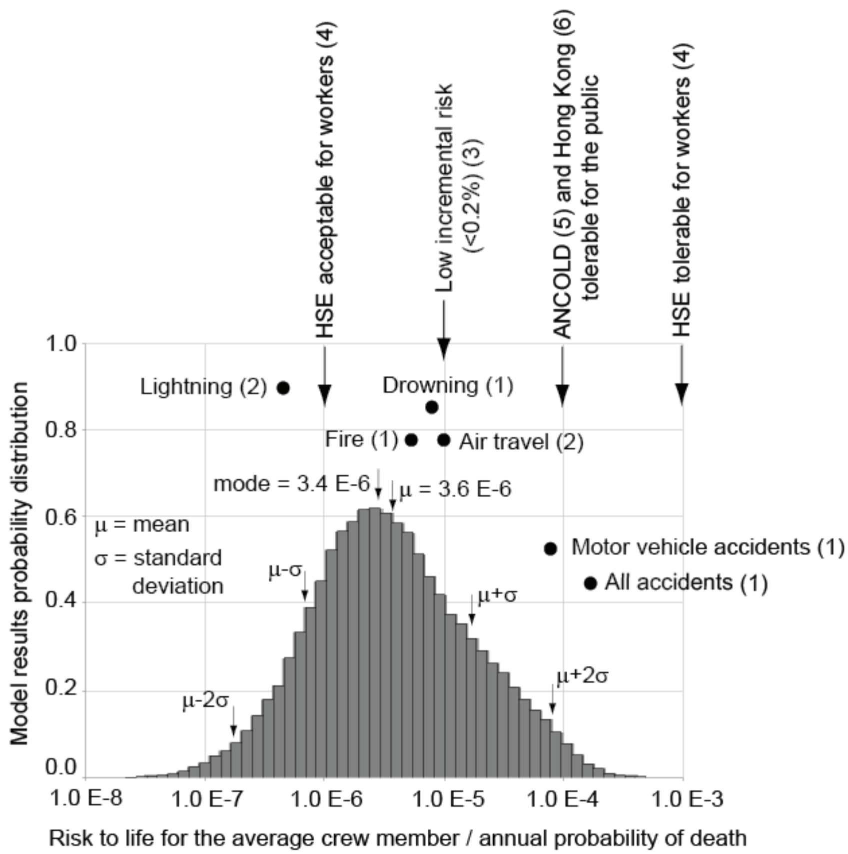
Figure 3-24 Individual risk to life for the average crew member and estimated individual risk for the worker most exposed - CP's Cascade subdivision study.

Figure 3-25 shows the distribution of the risk to life for the average crew member estimated in the second QRA. also shown are the selected risk evaluation criteria and common risks. The mean and mode of the estimated individual risk PDF are 3.6 x 10^{-6} and 3.4 x 10^{-6} respectively.

3.4.3. Risk Evaluation

Society's perception of risk varies between different regions depending on the social, cultural and economic context. Society's risk tolerance also varies (Morgenstern 1995, Finlay and Fell 1997). The risks estimated in this chapter are compared against widely used risk evaluation criteria. Even though these criteria were derived for other locations and contexts, they are considered applicable for illustrative purposes.

The individual risk evaluation criteria selected for comparison were those developed for people living in landslide prone areas in Hong Kong (ERM 1998), risks associated with dam failures in Australia (ANCOLD 2003), and the criterion proposed for land use planning around industries in the UK (HSE 2001). This last one was included given its wide spread application and because it proposes risk criterion for workers.



(1) Derived from the 2007 age-standardized mortality rates for the Canadian population (Statistics Canada 2010). (2) Data from Baecher and Christian (2003). (3) Porter *et al*. (2009) suggestion that the incremental risk is low if it doesn't exceed 0.2% of the Canadian age-standardized risk of loss of life. (4) HSE (2001). (5) ANCOLD (2003). (6) ERM (1998).

Figure 3-25 Monte Carlo simulation output for the risk to life of the average crew member estimated in the second QRA - CP's Cascade subdivision study.

The risk estimated in the initial QRA (Figure 3-24) for both the average and the most exposed crew member, and the mean and mode of the risk probability distribution obtained with the second QRA (Figure 3-25) is well below tolerable limits set for workers (HSE 2001) and below tolerable limits set for the public (ERM 1998 and ANCOLD 2003). Figure 3-25 also shows that the risk value corresponding to the mean plus two standard deviations (97.7% of results are lower than this value) is also below the tolerable individual risk criteria selected.

The International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) Technical Committee on Risk Assessment and Management proposed a Glossary of Terms for Risk Assessment. It stated that individual risk to life is the increment of risk to the individual in addition to the everyday risk if the hazard was not present (Fell *et al*. 2005). In this regard, Porter *et al*. (2009) estimated that an increase in individual risk of 1×10^{-5} would represent an increase of less than 0.2% over the average Canadian risk to life, which could be considered as low. It is noted that the central tendency estimated for the individual risks plot below this value.

The estimated risks, however, lie above the acceptable limits set for workers by the HSE (2001). These evaluation indicates the risks at the section analysed lie within the ALARP zone (As Low As Reasonably Practical) and measures are required to minimize the risks posed by the slope hazard, as long as the benefits outweigh the cost of mitigation. This conclusion was not unexpected, for this section is a highly hazardous one, where considerable risk mitigation is active (rock fall detection fences, ditch cleaning, scaling), thus complying with the ALARP principle.

3.5. CONCLUSION

The risk related to slope failures along miles 2 through 15 of CP's Cascade subdivision was estimated by means of two approaches. Both QRAs followed the same ETA logic, however population of them differed. The initial QRA selected best estimate values for the input variables that required subjective probabilities to be elicited, while the second QRA elicited believable upper and lower limits for these values.

The estimated risk probability distribution for the second QRA showed normality when plotted in semi-logarithmic scale. Calculation of the mean and standard deviation was done for the base-10 logarithm of the model output. This method minimizes the influence of the output values when calculating the point estimates (central tendency and variability) and treats each order of magnitude as a risk category. This approach is compatible with how risk is perceived when dealing with several orders of magnitude. It is also compatible with how risk evaluation criteria are expressed, while further allowing for the uncertainty in the estimated risk to be measured.

Unmeasured uncertainty associated with eliciting the upper and lower subjective probability limits is still present in the result. However, this uncertainty is considered much smaller than that related to a single value of risk. It is noticed that other sources of uncertainty, such as model uncertainty, can still represent the major source of error. In this regard, the validity of the model can be assessed to a certain extent by comparing the model partial outputs against available data. In this study, the model derailment probability was compared against derailment statistics in the section. This also opens the possibility for model calibration and upgrade in light of new data

Figure 3-26 shows the approach adopted for the second QRA in parallel with a simplified flowchart of the QRA framework. The process model is built for the hazard and consequence analyses, which include defining values for the input parameters and their variability (upper and lower limits and defining a PDFs). Risk is then estimated through a Monte Carlo simulation and results presented as a PDF of the estimated risks, plotted in semi-logarithmic scale.

Once the risk has been quantified, QRA can assist in defining the best strategy to comply with the organization's safety objectives. Furthermore, analysis of each step of the QRA process can highlight where mitigation efforts should be placed.

The low estimated individual risks in this analysis correspond to the short period of time each individual spends in the study section. The total risk (probability of fatality) is distributed through a large number of employees rendering low individual risks. If reduction of the total risk was to be considered, mitigation measures should focus on lowering the probability of one or more fatalities. Site inspections, scaling works, rock fall detection fences, ditch maintenance and protective walls; are all in place at the site. These either reduce the hazard frequency or the consequence probability and hence reduce the total risk in the area.

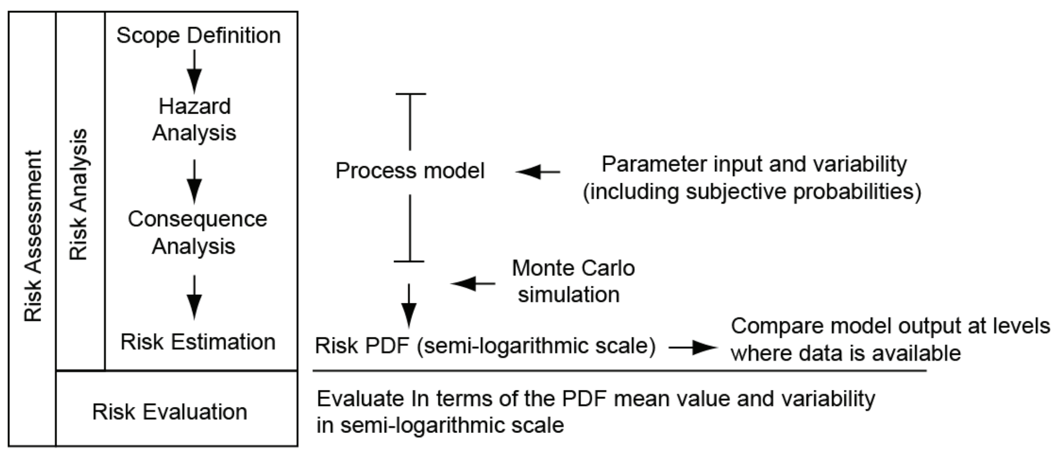


Figure 3-26 Diagram of the QRA approach adopted in the second QRA (right) in relation to a simplified flowchart of the QRA framework (left) - CP' s Cascade subdivision study.

A simple, yet comprehensive analysis has been presented, readily applicable by the practitioner and organizations. This analysis is intended to demonstrate the QRA methodology applied to a section of track that has a substantial slope failure database. QRA for such conditions is shown to be a valuable tool for decision making. Reviews of the analyses should be carried out at regular intervals, and probabilities updated in light of new information. As more experience is gained with the application of the QRA process, it may prove to be a suitable tool for risk management of all aspects of the railway operation.

CHAPTER 4: QRA OF A ROCK SLOPE UNDERGOING EXTREMELY SLOW DEFORMATIONS

This chapter presents a brief description of the rock slope, as well as an initial analysis of the likely failure scenarios and a Failure Modes and Effects Analysis (FMEA). This chapter further deals with the development of a quantitative risk assessment (QRA) for the Checkerboard Creek slope with respect to the risks associated with a potential flood of the town of Revelstoke.

4.1. INTRODUCTION

The Checkerboard Creek rock slope is located 1.5 km upstream of Revelstoke Dam, on the eastern slope of the Columbia River Valley. A network of active tension cracks was discovered shortly after completion of the Revelstoke Dam in 1983 and detailed investigation and monitoring was initiated. These investigations revealed that the tension cracks were associated with an extremely slow-moving rock mass with no through going basal shear zone. Stewart and Moore (2002) concluded that the deformations were consistent with disaggregated rock mass dilation and rotation mechanisms. Moreover, the monitoring data revealed an annual displacement cycle of about 10 mm with movements beginning in October, as the near ground surface temperature decreases, and ceasing in April / May, when the ground begins to warm up (Watson *et al*. 2004).

The importance of the Checkerboard Creek rock slope stability conditions is related to its location within the Revelstoke Dam reservoir, and to a lesser extent the existence of a secondary highway along its toe (Highway 23 - see Figure 4-1). The consequences of a potential slope failure and subsequent wave generation within the reservoir would compromise the earth and concrete dam structure, as well as the power house, and potentially flood downstream populated areas.

4.1.1. Checkerboard Creek Geometry and Boundaries

The Checkerboard Creek rock slope has a height of approximately 260 m from Highway 23, at an elevation of about 590 m, to the middle reach of Checkerboard Creek, at an average elevation of 850 m (see Figure 4-1). The width of the slope is about 600 m. The overall slope angle is about 30 degrees, being steeper at the toe (45 degrees) and flatter in the upper area (25 degrees) (Watson *et al*. 2004). The extent of the deforming rock mass has been interpreted from geological studies and deformation monitoring. The upper boundary is well defined by the alignment of the uppermost exposed tension cracks.

Part of this chapter was published in: Macciotta, R., Cruden, D.M., Martin, C.D. and Morgenstern, N.R. 2011. Combining geology, morphology and 3D modelling to understand the rock fall distribution along the railways in the Fraser River Valley, between Hope and Boston Bar, B.C. International Symposium on Rock Slope Stability in Open Pit Mining and Civil Engineering, Vancouver, BC, Canada, September 2011.

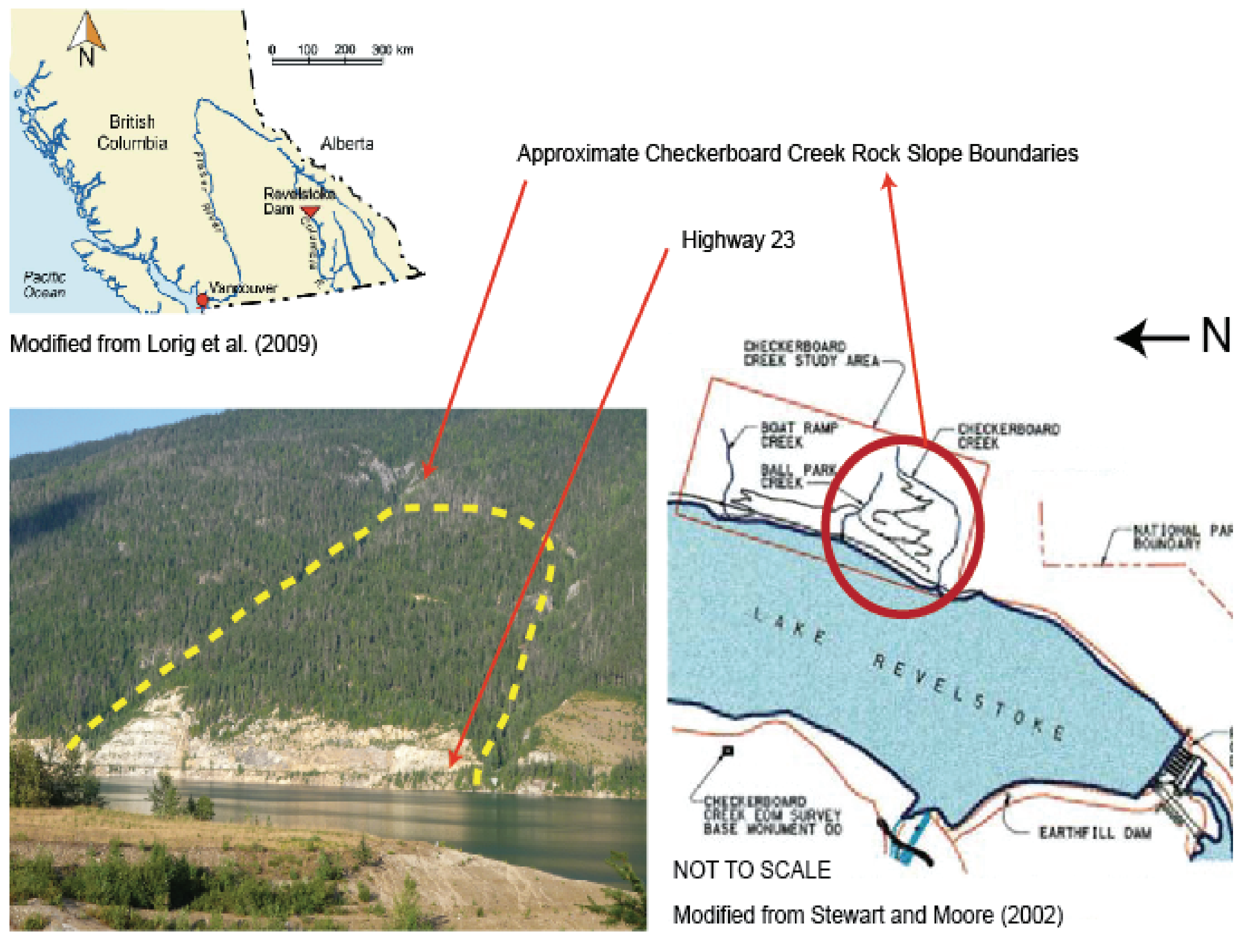


Figure 4-1 Location and approximate boundaries of the Checkerboard Creek rock slope.

The lateral boundaries, as well as the toe boundary are not as clear and have been interpreted from the site geology, slope topography and deformation patterns. The active zone has an average slope angle of approximately 45 degrees, being steeper at the toe (road cut) with a slope angle of 50 – 60 degrees. Deformations have been detected up to 50 - 60 m deep. This active zone has a total volume estimated to range between 2 to 3 Million m^3 (Watson *et al*. 2004). Figure 4-1 shows the location and approximate boundaries of the Checkerboard Creek rock slope.

4.1.2. Geology of Checkerboard Creek Rock Slope

The Checkerboard Creek rock slope comprises massive to weakly foliated granodiorite overlying the easterly dipping Columbia River Fault, which has developed a broad zone of altered and mechanically deformed rock. Shears and joints in the area dip steeply into and out-of slope at angles of 60 to 90 degrees from horizontal. The rock mass quality ranges from very strong, fresh, undisturbed and blocky rock to highly weathered and altered, weak and disturbed rock. Sheared and crushed zones are commonly found. The poor quality rock mass is typically found within 60 m from the slope surface, where the active deformations have been observed. Rock mass beneath this area is generally fair to good in quality, with localized zones of poor quality rock along shear zones and sub-vertical joints (Stewart and Moore 2002). Figure 4-2 illustrates the Checkerboard Creek rock slope geology. The groundwater regime within the Checkerboard Creek slope is inferred from piezometric data during drilling, monitoring of piezometer arrays and observations during site inspections. These have revealed numerous, discrete, pore pressure differences of up to 40 m across short lengths which is indicative of a compartmentalized groundwater regime. It is understood this compartmentalized groundwater regime corresponds to the low permeability materials found along the shear zones. Continuously saturated conditions have been observed 50 to 80 m below the surface. These depths are deeper than the observed extent of the displacing rock mass. Seasonal variations in piezometric levels of up to 20 m occur, mainly at the top of the continuously saturated rock mass, and diminishing with depth (Stewart and Moore 2002).

4.1.3. Monitoring of the Checkerboard Creek Rock Slope and Interpreted Deformation Patterns

The slope is being monitored by an array of surface and sub-surface instrumentation. Parameters considered in the monitoring system include displacements, water pressures and temperature within the rock mass, and air temperature and precipitation in the area. The instrumentation layout allows for monitoring of the overall moving mass as well as areas outside the deforming mass and areas down slope of the large tension cracks at elevation 700 m (considered a critical area). An automatic data acquisition system provides near real-time monitoring data of selected instruments, which are constantly reviewed at Revelstoke Dam (Stewart and Moore 2002, Watson *et al*. 2004).

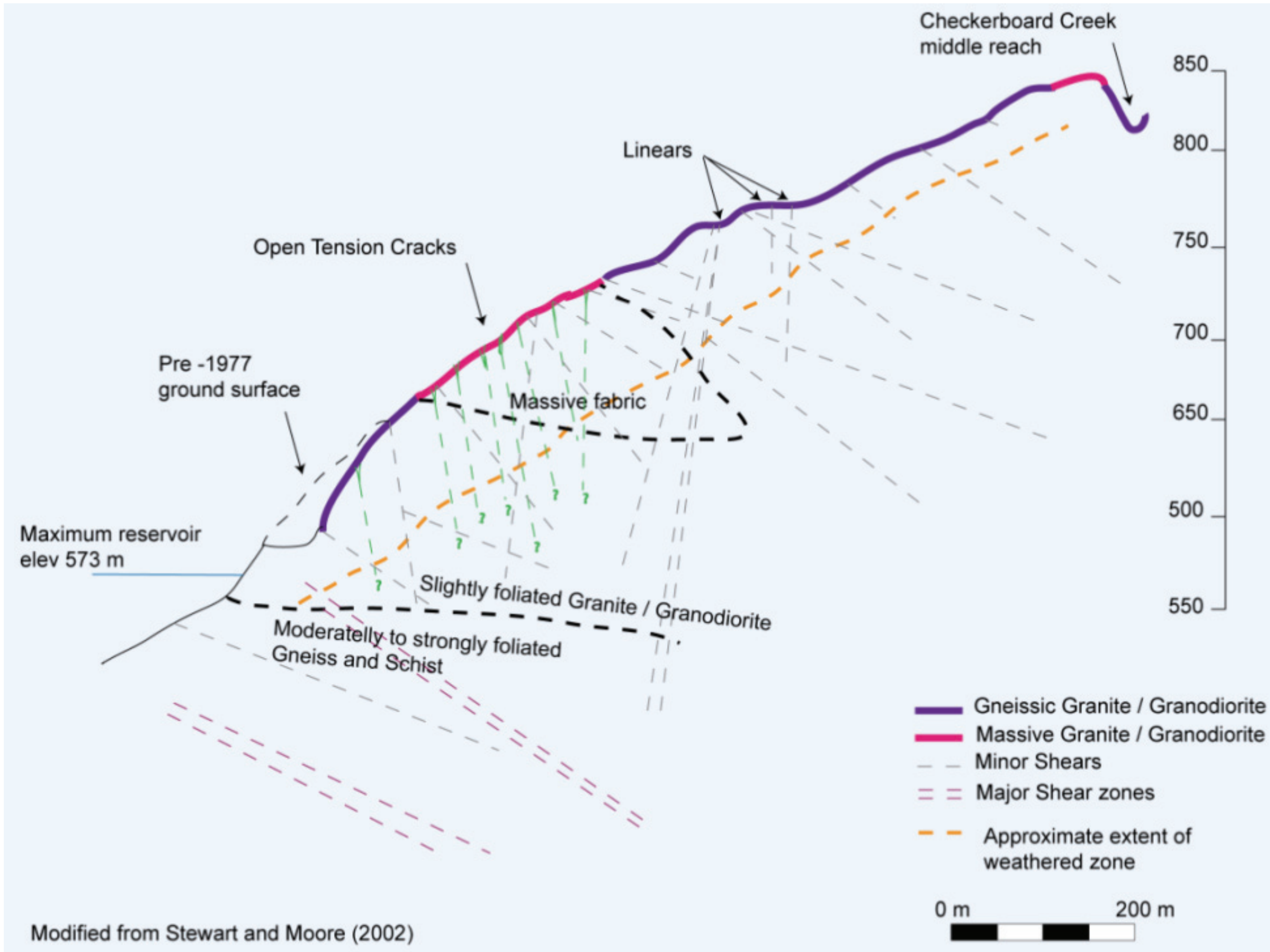


Figure 4-2 Simplified sketch of the Checkerboard Creek rock slope geology.

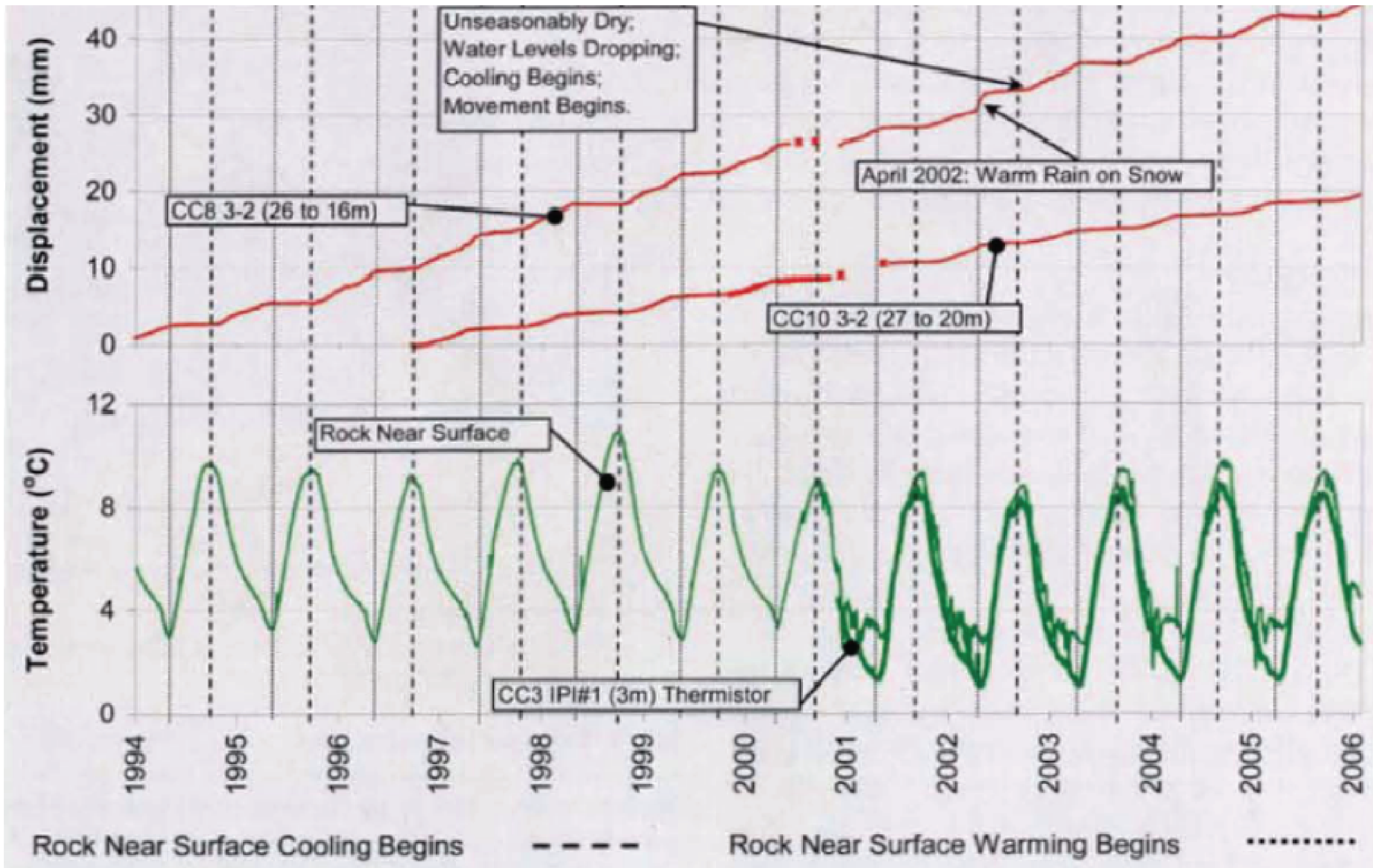
Displacement monitoring has revealed an annual displacement cycle dominated by an active period from early October to April/May (early autumn throughout late winter), and a relatively quiet period from May to September (spring and summer). The displacement rate of the deforming rock mass is 0.5 to 13 mm/y, being greatest at the surface and decreasing progressively with depth up to a point where no deformation is detected (about 55 m below surface). The deformations are generally widely distributed within the deforming mass, however there are zones where these are more concentrated or absent. These patterns indicate that deformations are distributed within the entire rock mass (Watson *et al*. 2004) rather than sliding as a block through a continuous failure plane (Stewart and Moore 2002).

4.1.4. Interpretation of Deformation Mechanisms at the Checkerboard Creek Rock Slope

Numerical analyses using FLAC (Fast Lagrangian Analysis of Continua) and UDEC (Universal Distinct Element Code) from Itasca Consulting Group, were used to aid in understanding the mechanisms and processes involved in the slope deformation pattern. (Stewart and Moore 2002, Watson *et al*. 2004).

With the aid of these models, the information gathered from site investigations, and the ongoing monitoring of the slope; the mechanisms leading to slope deformation were interpreted. Even though there are some indications of transient water pressures developing within the deforming mass and piezometric levels below the deforming zone raising during the active displacement periods, the annual cycle is strongly correlated to seasonal temperature variations in the bedrock near the surface (Figure 4-3). At the onset or acceleration of movement, and during the active displacement period, the near surface bedrock temperature is decreasing. During the inactive months, the near surface bedrock temperature is increasing (Watson *et al*. 2004).

Data from sub-surface thermistors indicate that these temperature fluctuations penetrate only about 10 m below the surface and are negligible beneath that depth, whereas the extent of the deforming rock mass is estimated to extend over 50 m in depth. Moreover, temperature changes at depth considerably lag those at the near surface. However, detailed numerical analysis simulating the seasonal temperature fluctuations indicates that the induced deviator stresses produce displacements deeper than the temperature fluctuation depth. Moreover, deformation patterns and magnitudes are consistent with the observations on site. It has been postulated that cooling of the near surface bedrock induces a reduction in the effective normal stress on sub-vertical discontinuities sub-parallel to the slope contours. This results in outward and downward displacement of the slope. During warming periods, the normal stresses increase and prevent further slipping (Watson *et al*. 2004).



After Watson et al. 2006

Figure 4-3 Deformation of the Checkerboard Creek rock slope compared to seasonal temperature variations in the bedrock near the surface and water levels.

4.1.5. Predictive Analyses

Watson *et al*. (2006) calibrated the UDEC model with the observed slope behaviour and conducted sensitivity predictive analyses. Results indicate that the slope would remain stable under extreme conditions of pore water pressure increases and extreme seismic events. Slope collapse could only be obtained in the models by a significant reduction of the rock mass strength or increases in pore water pressures beyond those deemed reasonably possible.

The models also identified a zone of less than 0.5 million m^3 above the highway cut that was the most likely failure scenario under extreme seismic loading conditions (Watson *et al*. 2004, Watson *et al*. 2006).

Several landslide impulse wave generation studies were carried out to assess the overtopping potential of the earth fill dam after a slope failure entering the reservoir. These studies included detailed and comprehensive physical wave-model of the reservoir slopes and dam. A detailed UDEC model was used to obtain the failed mass velocity, travel distance and nose shape (Lorig *et al*. 2009). The model evaluated several conditions for energy dissipation of the failing mass entering the reservoir in order to obtain a range of velocities and travel distances. The observations from the physical wave tests indicated that negligible overtopping of the earth fill dam occurred for any of the test conditions. For the worse case tested (1.2 Mm^3 slide falling at the highest velocities) there was less than 1 m of short duration overtopping of the earth dam about 200 m along the crest. Directly across from the slide, waves reached a maximum height of about 38 m above reservoir level while 2.9 km upstream from the dam, waves reached about 7 m above reservoir level (Watson *et al*. 2006).

4.2. QUALITATIVE ASSESSMENT OF SLOPE FAILURE PROBABILITY AND ITS POTENTIAL CONSEQUENCES

A comprehensive analysis of the risks related to the presence of the Checkerboard Creek rock slope requires assessing all the potential failure scenarios, the elements at risk, the magnitude of the potential losses and their probability. In this section, the potential failure scenarios and their likelihood are qualitatively estimated on the basis of the available information. Also, the exposed elements in the area are defined considering those that would imply the loss of life, given a failure scenario is realized. Finally, a FMEA is developed.

4.2.1. Failure Scenarios

Table 4-1 summarizes various failure scenarios based on the information obtained from site investigations, monitoring data analyses and numerical modelling. These scenarios are differentiated by the volume of rock involved in the slope failure and are considered for a project life of 100 years. Also shown in Table 4-1 is their perceived likelihood of occurrence. This perceived likelihood of occurrence is based on a preliminary qualitative review of the available data. No probability is assigned to the qualitative descriptor as that is what is sought from developing a formal QRA for the Checkerboard Creek rock slope. Justifications of these likelihoods are also presented in the table.

Scenario	Perceived likelihood	Justification
Rock falls (from small 1 m^3 to ranges of 10 to 100's m^3)	The most probable scenario (this scenario is considered likely to occur)	Justified by the rotational nature of movements and rock mass degradation mainly at the slope face above the highway cut and below the open tension cracks. Numerical models show the area as the most sensitive within the slope. Pavement scars from rock fall events less than 1 m^3 are present on the site.
Rock topples and falls less than 0.5 Mm^3 (highway cut – rock slope toe)	Probable scenario given slope failure occurs (this scenario is considered possible)	Defined by the most active deforming zone at the toe of the slope (highway cut) and backed up by numerical models. Its continuous deformation related to slope dilation makes this a probable scenario given a slope failure occurs.
Sudden release of 2 to 3 Mm^3 (actively deforming rock mass)	Unlikely	Defined by the total deforming zone interpreted from morphological evidence and instrumentation data. Includes zones where deformation rates are limited (2 – 5 mm/y) when compared to the most active zones (10 – 15 mm/y) and would require sudden strength loss of the entire area. Numerical models indicated stable conditions even under the 10 000 year return period seismic event.
Release of 20 – 55 Mm^3 (Checkerboard Creek rock slope)	Very unlikely to extremely unlikely	Defined by the morphology of Checkerboard Creek rock slope considering diverse depths of slope failure. No morphological evidence of active movement or recorded by instrumentation. Numerical models indicate stable conditions. Would require significant strength reduction not considered realistic within the next 100 years.

Table 4-1 Potential rock slope failure scenarios (100 year period) - Checkerboard Creek rock slope.

Subsequent analysis of the risks related to the Checkerboard Creek rock slope will be based on these failure scenarios and their potential consequences.

4.2.2. Elements at Risk

Elements at risk include the highway at the slope toe and its users, the Revelstoke dam and associated structures, populated areas downstream of the Revelstoke dam and recreational areas and activities within the reservoir (camping areas, boaters, tourists). An exhaustive analysis of the possible consequences requires knowing the location of the structures and the costs related to repairing/rebuilding them, as well as the financial losses associated with the disruption of their serviceability. Assessing the consequences to life requires knowledge of the number of people at every location. This includes the populated areas, traffic through the highway, campers, boaters, and how these are distributed throughout the year (temporal probability). Other aspects such as environmental losses and public perception also have to be considered.

The present study focuses on the risk to life related to a failure of the Checkerboard Creek rock slope. Figure 4-4 shows the areas at risk identified where consequences of a slope failure include the potential for life loss. The elements at risk are then grouped in:

- Highway 23,
- Revelstoke Dam and area, with associated structures (powerhouse, offices, tourist facilities),
- Martha Creek Provincial Park,
- Boat launch and picnic area north of Checkerboard Creek rock slope, and,
- City of Revelstoke and area, including the Revelstoke Airfield.

4.2.3. Failure Modes and Effect Analysis

The FMEA is intended to aid in the identification of the potential failure modes, exposed elements and the potential consequences. An assessment of the pre-failure signs and potential early detection is also included in the FMEA. Note the qualitative likelihood descriptor of the failure mode is taken from Table 4-1. Also, a preliminary relative severity of the consequences is presented to aid in the identification of the most critical failure modes. These relative severity is to be further resolved by formal QRA.

An exhaustive FMEA is required to facilitate a comprehensive QRA. The FMEA should account for all realistic failure modes and their consequences. **Table 4-2** shows the FMEA applied to the Checkerboard Creek rock slope.

4.2.4. Qualitative Risk Assessment of the Checkerboard Creek Rock Slope

Table 4-3 presents the qualitative risk assessment of the Checkerboard Creek rock slope. The assessment is presented as a matrix with one entry being the perceived likelihood of occurrence for each failure scenario and the other entry being the perceived severity of their associated potential consequences. As per tables 10 and 11, the assessment considers a 100 year period. Each scenario perceived likelihood and severity of potential consequences are taken from tables 10 and 11.

Assessment of the risk level for each scenario is done by colour coding the combinations of likelihood-severity in the matrix in Table 4-3. Extremely Unlikely and Very Unlikely likelihoods with Non Severe consequences associated with them were considered to pose negligible risks. Non severe consequences with higher likelihood of occurrence were considered to pose risks within acceptable limits (colour green in Table 4-3).

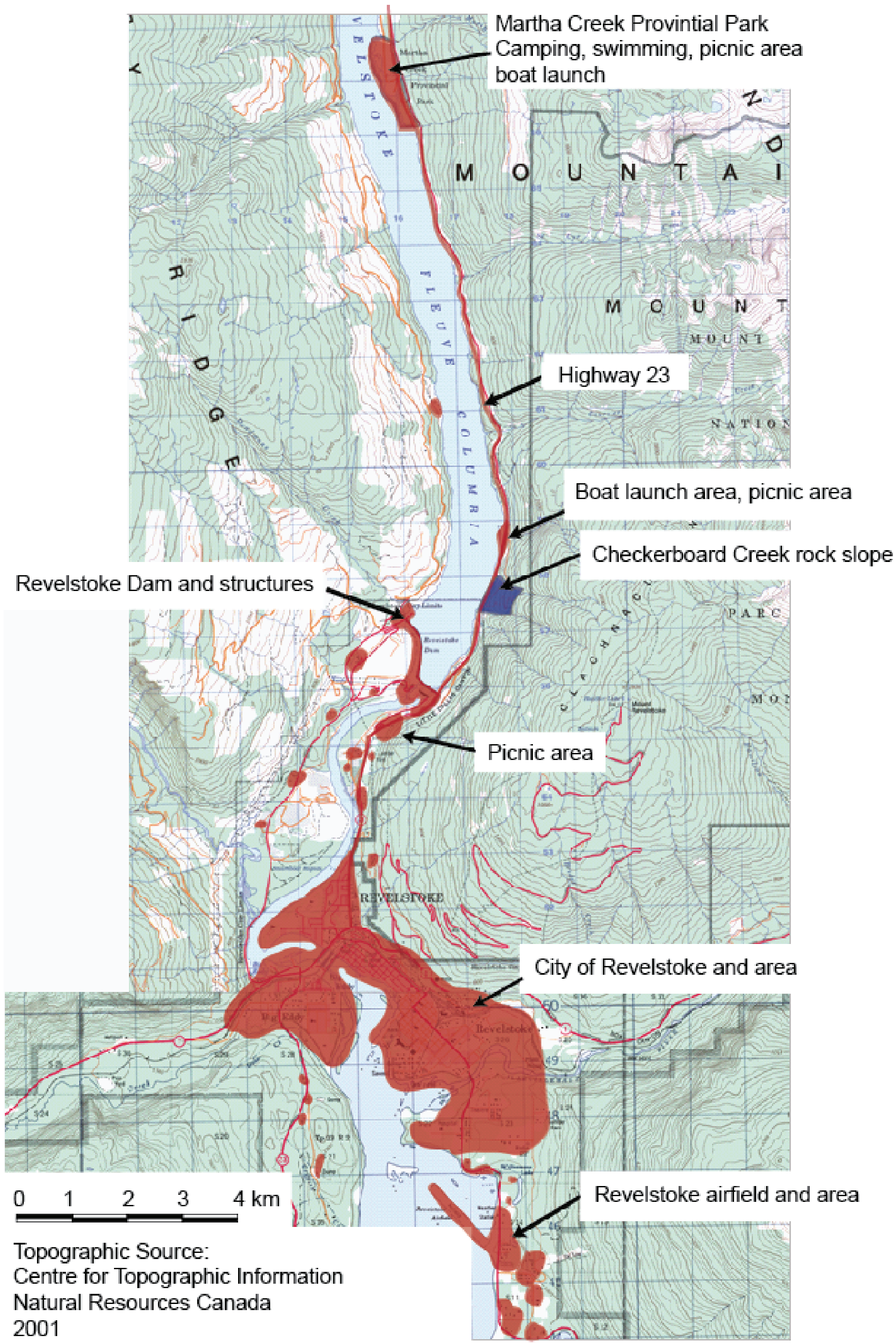


Figure 4-4 Areas at risk identified where consequences of failure of the Checkerboard Creek rock slope include the potential for life loss.

Failure Mode	Likelihood	Failure Early Detection	Potential Direct Effect	Potential Indirect Failure Effect and Wave Generation Effect	Perceived Severity
a) Rock falls	The most probable scenario (this scenario is considered likely to occur)	Visual inspection of unstable blocks (limited effectiveness). Likely to be triggered by precipitation events and seismic events.	Damage / blockage of Highway 23. Highway users killed and injured..	No wave generated. Economic loss due to highway serviceability being interrupted.	Non severe
b) Up to 0.5 Mm^3 (highway cut - rock slope toe)	Probable scenario given slope failure occurs (this scenario is considered possible)	Increase in rock fall events (highway cut). Acceleration of monitored deformations and possible change in annual displacement cycle. Rock mass deterioration (disaggregation and dilation) likely to be noticeable. Opening of new tension cracks and widening of existing ones within the deformation zone.	Severe damage and blockage of Highway 23. Instrumentation loss. Highway users killed and injured.	Wave generated. Potential damage to recreational areas and tourists (boaters, camping areas, hiking trails) up to limited height within the reservoir shoreline. Minor / negligible damage to the Revelstoke dam infrastructure (earth and concrete dam, spillway, powerhouse, access) and possible minor injuries / life loss for workers / tourists on site. Economic loss due to highway serviceability being interrupted. Economic loss due to hydroelectric plant serviceability being interrupted for limited period (precautionary). Very unlikely, minor damage at Revelstoke due to Columbia River wave after minor overtopping of dam. People injuries, structure damage and agricultural land loss related to it. Public concern and deterioration of relationship between BChydro and general public.	Slightly severe

Table 4-2 Checkerboard Creek rock slope Failure Modes and Effects Analysis (FMEA)

Failure Mode	Likelihood	Failure Early Detection	Potential Direct Effect	Potential Indirect Failure Effect and Wave Generation Effect	Perceived Severity
c) 2 to 3 Mm^3 (actively deforming rock mass)	Unlikely	Increase in rock fall events (highway cut). Acceleration of monitored deformations (larger area than case b). Likely change in annual displacement cycle. Possible development of a basal zone of sheared material leading to changes in the deformation trends with depth. Opening of new tension cracks and widening of existing ones within the deformation zone. If a basal shear zone develops, a back scarp and surface features are likely to be noticed.	Severe damage and blockage of Highway 23 (larger extent than case b). Instrumentation loss (larger extent than case b). Highway users killed and injured.	Wave generated. Potential damage to recreational areas and tourists (boaters, camping areas, hiking trails) within the reservoir shoreline. Worse than case b. Limited damage to the Revelstoke dam infrastructure (earth and concrete dam, spillway, powerhouse, access) and injuries / life loss of workers / tourists on site. Worse than case b. Economic loss due to highway serviceability being interrupted. Longer period than for case b. Economic loss due to hydroelectric plant serviceability being interrupted for limited period (precautionary). Minor scars of rock failure and wave generated along the shoreline of the reservoir and Columbia River downstream of dam (environmental concern). Serious public concern and deterioration of relationship between BChydro and general public. Might lead to stricter / modification of regulations. Worse than case b. Minor damage at Revelstoke due to Columbia River wave after dam overtopping. – People injuries / life loss, structure damage and agricultural land loss related to it.	Severe

Table 4-2 Checkerboard Creek rock slope Failure Modes and Effects Analysis (FMEA) - continued

Failure Mode	Likelihood	Failure Early Detection	Potential Direct Effect	Potential Indirect Failure Effect and Wave Generation Effect	Perceived Severity
d) 20 - 55 Mm^3 (entire slope)	Very to extremely unlikely	Instrumentation outside the active zone would start showing some displacement. Opening of new tension cracks and development of new surface features related to the new displacements.	Severe damage and blockage of Highway 23 (considerably larger extent than case c. All Instrumentation loss. Highway users killed and injured.	Wave generated. Damage to recreational areas and tourists (boaters, camping areas, hiking trails) within the reservoir shoreline. Considerably worse than case c. Severe damage to the Revelstoke dam infrastructure (earth and concrete dam breach, spillway, powerhouse, access) and injuries / life loss of workers / tourists on site. Considerably worse than case c. Economic loss due to highway serviceability being interrupted. Longer period than for case c. Economic loss due to hydroelectric plant serviceability being interrupted indefinitely. Severe damage at Revelstoke due to Columbia River wave and flood after overtopping / failure of dam – people injuries / life loss, structure damage, agricultural and recreational land loss related to it. Considerably worse than case c. Scars of rock failure and wave generated along the shoreline of the reservoir and Columbia River downstream of dam (environmental concern). Considerably worse than case c. Major public concern and deterioration of relationship with BChydro. Would lead to stricter regulations. Worse than case c.	Catastrophic

The term "worse than" used to compare effects between scenarios, is meant to indicate larger negative consequences to be expected from larger volume slope failures.

Table 4-2 Checkerboard Creek rock slope Failure Modes and Effects Analysis (FMEA) - continued

		Perceived Magnitude of Potential Consequences			
		Non Severe	Slightly Severe	Severe	Catastrophic
Perceived Likelihood	Extremely Unlikely				20 to 55 Mm^3
	Very Unlikely				
	Unlikely			2 to 3 Mm^3	
	Possible		Up to 0.5 Mm^3		
	Likely				
	Very Likely	Rock falls			

Table 4-3 Qualitative Risk Assessment of the Checkerboard Creek rock slope.

Risks posed by Slightly Severe consequences but Unlikely or less probable, and Severe consequences but Extremely Unlikely, were also considered within acceptable limits. Risks posed by Possible to Very Likely, Slightly Severe consequences; Unlikely and Very Unlikely, Severe consequences; and Very to Extremely Unlikely, Catastrophic consequences; were considered as approaching the limits of risk tolerance (Yellow colour in Table 4-3). Risks associated with Possible, Severe consequences and Possible to Unlikely, Catastrophic consequences were considered as requiring action to mitigate these risks (Orange colour in Table 4-3). Likely to Very Likely, Severe and Catastrophic consequences are consider to pose risks that need to be immediately mitigated (Red in Table 4-3).

This qualitative analysis synthesizes the potential failure scenarios and their potential consequences in a comprehensive manner. The knowledge of the actual and potential future slope behaviour is translated to potential slope failure scenarios and their perceived likelihood of occurrence. Knowledge of the elements at risk in the area and consideration of the secondary processes that can be triggered by a slope failure leads to a comprehensive count of the foreseeable consequences and their perceived magnitude or severity. This synthesis then allows for a qualitative assessment of the risks posed by the different slope failure scenarios, and what is the perceived urgency for mitigation measures.

Chapter 1 discussed some of the limitations of quantitative risk analyses and assessments. However, the analysis presented here was considered an essential step prior to the development of the QRA for the Checkerboard Creek slope. The analyses required for a QRA to be properly populated can be numerous and time consuming. In that regard, the qualitative analysis of failure scenarios and potential consequences, as well as the assessment of the associated risks, provide a road map for the QRA to be comprehensive and to focus on the scenarios and consequences considered critical.

4.3. QUANTITATIVE RISK ASSESSMENT

A QRA was developed for the Checkerboard Creek slope on the basis of published studies. The complexity of the system required population of models with data and information not readily available or not developed for all potential failure scenarios. This was resolved by adopting simple approaches to acquire the necessary information (sliding velocities and dam overtopping volumes for all scenarios, dam robustness against overtopping, flooding levels at Revelstoke for different overtopping wave scenarios). It is acknowledged that more comprehensive analysis might be required to populate the model given the risk levels are found to be near limiting thresholds. This study was then considered an initial stage to screen these levels of risk. The QRA presented in this study is limited to the potential life loss at the Revelstoke area given a slope failure occurs leading to the generation of an impulse wave that could overtop, and potentially breach, the Revelstoke Dam.

4.3.1. Hazard Analysis - Annual Probability of Failure

A hazard analysis consists in the characterization of the potential dangers (landsliding) and estimation of their occurrence probability. A description of the Checkerboard Creek rock slope is presented at the beginning of this chapter. This description includes geometry, volumes, monitored behaviour, deformation mechanisms and potential failure scenarios. Further detail can be found in Stewart and Moore (2002) and Watson *et al*. (2004), which include the climatic conditions in the area. All of these are part of a proper characterization of the potential slope failure and are not repeated here.

Given the QRA focuses on the life loss at Revelstoke after a landslide-induced impulse wave overtops / breaches the dam, the scenario considering rock fall events in Table 4-1 is not analysed any further.

Failure of the Checkerboard Creek rock slope is defined from a serviceability point of view. It is considered the slope has failed when the amount of deformation is such that Highway 23 is blocked with material that could potentially enter the reservoir. The definition also considers that the deformation velocity should be equal or greater than rapid according to Cruden and Varnes (1996) velocity classification. This corresponds to our ability to successfully manage situations where slopes show slower deformations.

4.3.1.1. Annual Probability of Slope Failure - Initial Considerations and Estimations

The nature of the slope deformation mechanisms and lack of a continuous basal sliding surface makes it extremely difficult to estimate the slope failure probability based on numerical simulations under specific conditions. The uniqueness of the slope characteristics (geology, geometry and history of highway cut and reservoir infilling) also makes it difficult to correlate historical failure frequencies to the likelihood of failure of the Checkerboard Creek slope. Estimation of the failure probability for each scenario requires direct input of expert judgement on the basis of all the studies and knowledge summarized in previous sections.

The deformations observed at the Checkerboard Creek rock slope correspond to relative displacements of blocks in a highly disaggregated mass. The magnitude of the deformations decrease with depth down to about 50 to 60 m, where the rock mass has shown to be of higher quality. The change in the degree of disaggregation is not sharp, but appears to be gradual. Relative block displacements are related to progressive failure along multiple shear planes within the deforming mass and the continuous degradation of the rock mass with depth. Also, geomorphic evidence (mainly tension cracks) suggest these deformations have been occurring over a long period of time leading to surface displacements of 10 m or more, and numerical models (distinct element codes) indicate there is a considerable reserve of rock mass strength against large slope failures under static, seismic and elevated pore water pressure scenarios (Watson *et al*. 2006). These models, together with site inspections, suggest that the most vulnerable area of the slope is the road cut above Highway 23 (a volume of up to 0.5 Mm^3).

An inventory of historic rockslides across the Canadian Cordillera was used by Hungr and Evans (1993) to gain an idea of their occurrence probability order of magnitude. The geological and morphologic context of most of these slope failures, their mechanisms, and triggers are widely different from those at the study area. However, their findings can give some insight into the orders of magnitude of the return periods of rockslides in the Canadian Cordillera. Their analysis suggested that for events larger than 2 to 3 Mm^3 and events larger than 20 to 55 Mm^3, the historical cumulative occurrence probabilities were about 5×10^{-3} and 10^{-4} per year respectively. The areal extent associated with these likelihoods was 10 000 km^2. It is estimated that the Revelstoke reservoir and surrounding mountains represent an area of about 1000 km^2. Two slopes have been recognized to show the lowest stability conditions in this area (Downie and Checkerboard slides). This approach would suggest annual failure probabilities for the Checkerboard Creek slope in the order of 10^{-4} and 10^{-6} for events larger than 2 to 3 Mm^3 and events larger than 20 to 55 Mm^3, respectively.

These conclusions and the deformation patterns described in previous sections lead to the belief that the annual likelihood of failure of the entire slope (20 to 55 Mm^3) is very to extremely unlikely, that of the actively deforming rock mass (2 to 3 Mm^3) is unlikely, and that of the slope cut (up to 0.5 Mm^3) is probable. As such, preliminary subjective probabilities were elicited for the annual failure probability of the Checkerboard Creek rock slope scenarios (Table 4-4) .

Scenario	Subjective probability: annual probability of failure
Up to 0.5 Mm^3 (highway cut - rock slope toe)	10^{-3} to 10^{-1}
2 to 3 Mm^3 (actively deforming rock mass)	10^{-5} to 10^{-3}
20 - 55 Mm^3 (Checkerboard Creek rock slope)	Negligible to 10^{-6}

Table 4-4 Preliminary subjective probabilities elicited for the annual failure probability of the Checkerboard Creek rock slope.

4.3.1.2. Process Model to Estimate the Slope Annual Probability of Failure

The events leading to a failure of the Checkerboard Creek rock slope were modelled as a process with the aid of an event tree analysis (ETA). Figure 4-5 shows the ETA used in the analysis. Failure conditional probabilities were input as probability density functions (PDF), and the ETA was evaluated through a Monte Carlo simulation technique.

Slope Cyclic Behaviour

The first tree branch refers to the slope cyclic behaviour. This level can have two outcomes, warming or cooling period. The warming period, when no deformation is detected, lasts from May to September (5 months). The probability assigned is 5/12 = 0.417. For the cooling period (active deformation period of the slope) a probability of 0.583 is assigned.

Seismic Events

The second tree branch level corresponds to the possibility of a seismic event occurring in the area. The seismic hazard is characterized by Peak Ground Accelerations (PGA) of approximately 0.07 times the gravity (0.07g) and 0.14g for Annual Exceedance Frequency (AEF) of 1 in 475 years and 1 in 2 475 years, respectively (NRC 2010). Numerical models had been used to predict the stability conditions under seismic events (Watson *et al*. 2004). These indicate that even for the earthquake with 1 in 10 000 year return period, there is a reserve of rock mass strength against collapse for masses larger than 0.5 m^3. Three seismic scenarios were considered: 1) PGA < 0.05g indicating none to minor seismic event conditions, 2) 0.05g < PGA < 0.2g indicating moderate to severe seismic conditions accounted for in numerical models, 3) 0.2g < PGA indicating the potential for an extreme event. Assigned probabilities are associated with return periods of 1 in 475 for the 0.05g PGA and 1 in 10 000 for the 0.2g PGA. This last one as assumed by Stuart and Moore (2002).

Groundwater Level

The third tree branch level corresponds to the ground water conditions within the active zone of deformation. As previously discussed, transient, perched pore water pressures have been measured within the deforming mass, while the continuously saturated mass is beneath the active depth of deformation. Even when weather and piezometric records extend for over 30 years, there is no clear relation between precipitation and piezometric response, and the deforming mass has shown to be essentially drained. A hydrogeological model would be required to estimate the probability of an increase of the water table in a stochastic manner. However, the complexity of the disaggregated rock mass, and not having consistent pressure fluctuations within this mass, suggests the necessary information to develop a reliable and calibrated hydrogeological model of the slope is not available.

Subjective probabilities were elicited to populate this branch. It was assumed no changes occur in the groundwater behaviour for precipitation events or wet seasons with return periods of up to 1 in 50 years. This is considered the average condition. Increases in the groundwater table are expected to be moderate for events with return periods up to 1 in 500 years. A significant increase is associated with events or wet seasons with return periods over 1 in 500 years. It is recognized that further investigations and observations are needed to improve the assessment at this level of the ETA.

Some branches consider increases of the groundwater table in combination with a seismic event. It is necessary to consider the probability that the seismic event occurs at the time of the year where the increased groundwater table occurs. It was estimated that an increase in the groundwater table can potentially remain for about one month as an average. Then, the probability of an elevated groundwater table for those branches with a seismic event above PGA of 0.05g was reduced by one order of magnitude.

It is expected that increases in the groundwater table within a highly disaggregated, drained mass should require periods of time considerably wetter than average, rather than a short duration rainfall event. Table 4-5 shows the monthly average precipitation at Revelstoke. This data suggest that monthly precipitation has limited variation through the year. Another potential trigger for an increase in the groundwater table is the spring thaw. However, the active slope deformation period starts in October and the relation between spring thaw and deformation rate is not clear. It was then decided to apply the same subjective probabilities for increases of groundwater table for the warming and cooling periods.

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	TOT
108.7	85.6	65.8	55.4	58.3	73.3	65	63.1	58.8	79.8	108.9	123	945.5
11%	9%	7%	6%	6%	8%	7%	7%	6%	8%	12%	13%	

Table 4-5 Monthly average precipitation in mm at Revelstoke (Environment Canada 2012).

Conditional Probability of Slope Failure

Each volume scenario for each branch of the ETA has a corresponding conditional probability of slope failure. These conditional probabilities are based on the degree of belief for a failure to occur in light of the available information.

An upper and lower value were adopted to define the range of elicited subjective probabilities considered plausible. Assigning an equal likelihood throughout this range reflects the uncertainty in the nature of the probabilities being elicited. In a Monte Carlo simulation technique, a uniform probability distribution through more than one order of magnitude will tend to randomly select 10 times more samples from the higher magnitude than from the immediately lower magnitude. To have the simulation select a similar number of samples from each order of magnitude, a linear cumulative distribution in a semi-logarithmic scale was adopted within the failure probability ranges. Figure 4-6 shows an example of the linear cumulative distribution in semi-logarithmic scale adopted between the selected lower and upper values of 10^{-8} and 10^{-6} respectively. This distribution better reflects the notion of uncertainty related to elicited subjective probabilities when covering over one order of magnitude.

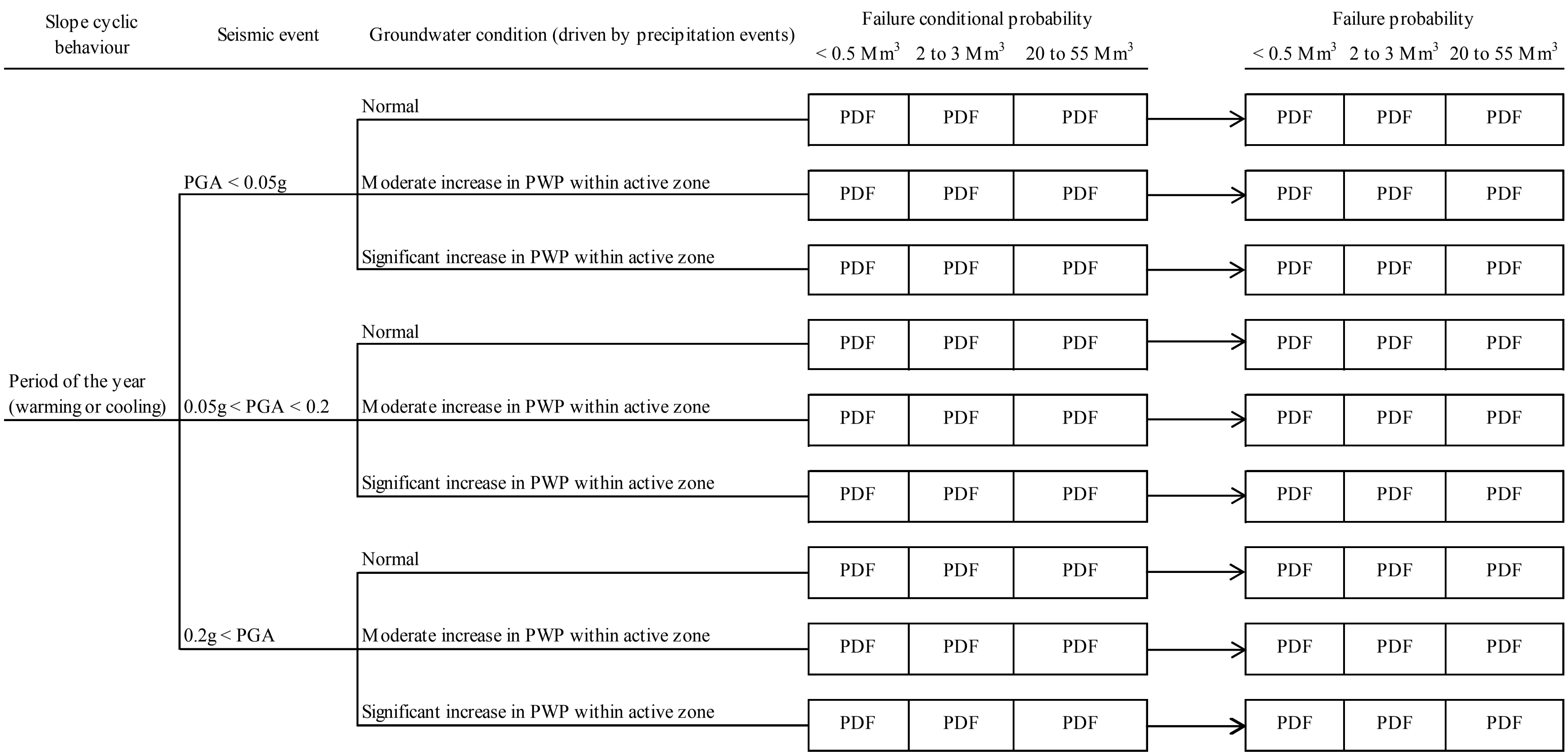


Figure 4-5 Event tree modelling the processes leading to a slope failure of the Checkerboard Creek rock slope.

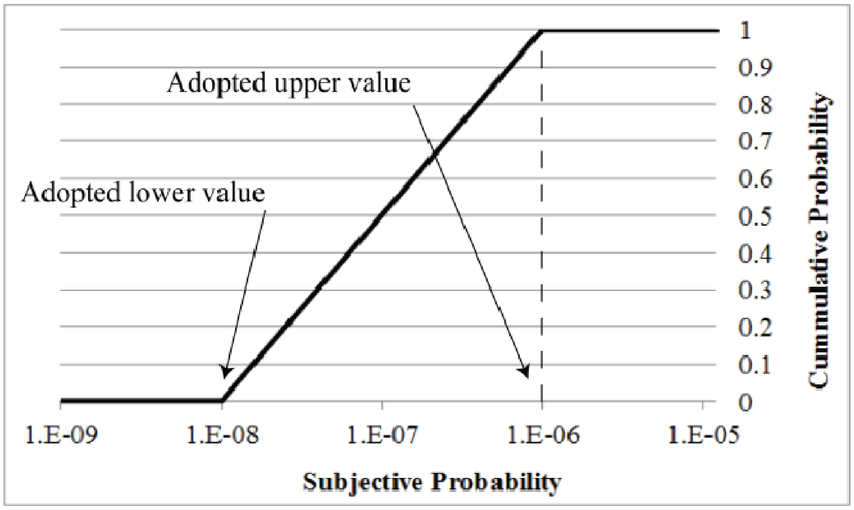


Figure 4-6 Example of the linear cumulative distribution in the semi-logarithmic scale adopted for the failure probabilities between the lowest and highest values obtained by expert elicitation processes.

Table 4-6 presents the lower and upper subjective probabilities elicited in light of the available information. Each iteration of the Monte Carlo simulation selects random values from the subjective probability ranges adopted, according to the PDF defined. It was decided to positively correlate all these input values (correlation of 0.8). This implies that if a high value is selected for an input variable, high values will tend to be selected for all other input variables. This was done to keep consistency between failure probabilities of different scenarios (volumes and triggers).

Failure Probabilities

The results of the Monte Carlo simulations were analysed in semi-logarithmic scale. This implies that calculations of mean values and percentiles are performed over their logarithms. This approach treats each order of magnitude as a probability category and minimizes the effect of their magnitude in the calculation of point estimates. Table 4-7 shows the estimated mean, median, minimum and maximum values for the slope failure annual probabilities. Also presented are the Percentiles 5% and 95% of the results. Figure 4-7 and Figure 4-8 show an example of one iteration of the ETA for the warming and cooling periods, respectively.

Warming period (quiet deformation period)							
Volume < 0.5 Mm^3		Seismic event					
		Minor		Moderate		Significant	
		Lower	Upper	Lower	Upper	Lower	Upper
Groundwater table increase	Normal	5.0E-04	5.0E-03	1.0E-03	5.0E-02	5.0E-01	1.0E+00
	Moderate	1.0E-03	1.0E-02	5.0E-03	5.0E-02	1.0E+00	1.0E+00
	Significant	5.0E-03	5.0E-02	1.0E-01	5.0E-01	1.0E+00	1.0E+00
Volume 2 to 3 Mm^3		Seismic event					
		Minor		Moderate		Significant	
		Lower	Upper	Lower	Upper	Lower	Upper
Groundwater table increase	Normal	1.0E-05	1.0E-04	1.0E-03	1.0E-02	1.0E-01	1.0E+00
	Moderate	1.0E-03	1.0E-02	1.0E-02	1.0E-01	5.0E-01	1.0E+00
	Significant	1.0E-02	1.0E-01	1.0E-02	5.0E-01	5.0E-01	1.0E+00
Volume of 20 to 55 Mm^3		Seismic event					
		Minor		Moderate		Significant	
		Lower	Upper	Lower	Upper	Lower	Upper
Groundwater table increase	Normal	1.0E-06	1.0E-06	1.0E-06	1.0E-04	1.0E-03	5.0E-02
	Moderate	1.0E-06	1.0E-06	1.0E-05	1.0E-03	1.0E-03	1.0E-01
	Significant	1.0E-06	1.0E-05	1.0E-05	1.0E-03	1.0E-03	1.0E-01
Cooling period (active deformation period)							
Volume < 0.5 Mm^3		Seismic event					
		Minor		Moderate		Significant	
		Lower	Upper	Lower	Upper	Lower	Upper
Groundwater table increase	Normal	5.0E-03	5.0E-02	1.0E-02	1.0E-01	5.0E-01	1.0E+00
	Moderate	1.0E-02	1.0E-01	5.0E-02	1.0E-01	1.0E+00	1.0E+00
	Significant	5.0E-02	5.0E-01	1.0E-01	5.0E-01	1.0E+00	1.0E+00
Volume 2 to 3 Mm^3		Seismic event					
		Minor		Moderate		Significant	
		Lower	Upper	Lower	Upper	Lower	Upper
Groundwater table increase	Normal	1.0E-04	1.0E-03	1.0E-03	1.0E-02	1.0E-01	1.0E+00
	Moderate	1.0E-03	1.0E-02	1.0E-02	1.0E-01	5.0E-01	1.0E+00
	Significant	1.0E-02	1.0E-01	1.0E-02	5.0E-01	5.0E-01	1.0E+00
Volume of 20 to 55 Mm^3		Seismic event					
		Minor		Moderate		Significant	
		Lower	Upper	Lower	Upper	Lower	Upper
Groundwater table increase	Normal	1.0E-06	1.0E-06	1.0E-06	1.0E-04	1.0E-03	5.0E-02
	Moderate	1.0E-06	1.0E-06	1.0E-05	1.0E-03	1.0E-03	1.0E-01
	Significant	1.0E-06	1.0E-05	1.0E-05	1.0E-03	1.0E-03	1.0E-01

Table 4-6 Lower and upper values elicited for the subjective probabilities of slope failure of the Checkerboard Creek rock slope.

Scenario	< 0.5 Mm3			2 to 3 Mm3			20 to 55 Mm3		
	Warming Period	Cooling Period	Annual	Warming Period	Cooling Period	Annual	Warming Period	Cooling Period	Annual
Mean (u)	7.26E-4	9.67E-3	1.04E-2	8.16E-5	2.79E-4	3.63E-4	8.36E-7	1.17E-6	2.03E-6
Median	7.25E-4	9.70E-3	1.05E-2	8.34E-5	2.85E-4	3.70E-4	7.27E-7	1.02E-6	1.83E-6
Minimum	2.37E-4	3.06E-3	3.30E-3	2.50E-5	8.63E-5	1.11E-4	4.59E-7	6.41E-7	1.10E-6
Maximum	2.24E-3	3.03E-2	3.25E-2	2.49E-4	8.61E-4	1.11E-3	2.60E-6	3.63E-6	6.23E-6
Percentile 5%	2.69E-4	3.47E-3	3.78E-3	3.01E-5	1.02E-4	1.34E-4	4.69E-7	6.56E-7	1.13E-6
Percentile 95%	1.97E-3	2.68E-2	2.85E-2	2.10E-4	7.39E-4	9.40E-4	2.19E-6	3.06E-6	5.03E-6

Table 4-7 Annual probabilities of failure for the Checkerboard Creek rock slope.

4.3.2. Consequence Analysis - Life Loss Probability for the Population at Revelstoke

Given a slope failure of a given volume occurs, the population in Revelstoke and area will be impacted if a sudden surge is generated after a wave overtops and/or breaches the dam. Figure 4-9 presents the process model developed to estimate the likelihood and magnitude of such event given a slope failure. In this model, the wave height will depend on the volume of material failed and its velocity when entering the reservoir. This wave height could then lead to a breach of the earthfill dam. The flood level at Revelstoke will depend on the wave height overtopping the dam or, if the earth fill dam is breached, the water flow through the breach. Combining these with the number of exposed population and their vulnerability, the number of fatalities and likelihood of the scenario can be estimated. The following subsections discuss the analyses and thinking behind the definition of the necessary input parameters for the consequence analysis. The concrete gravity dam was considered robust enough to withstand overtopping.

4.3.2.1. Slide Velocity

The slope debris entrance velocity after failure is denoted as Slide Velocity regardless of the failure mechanism involved. The velocity scale proposed by Cruden and Varnes (1996) was adopted in this study. A PDF over this scale was then elicited for the slide velocity. This function is based on the geological information, the slope's observed behaviour, and the published literature on predictive numerical models for the slope. The falling material was also modeled as a rigid block sliding on inclined planes (Körner 1976, Slingerland and Voight 1979) as shown in Figure 4-10. This helped bounding the potential failure velocities when entering the reservoir. Frictional strength losses are then assumed along the sliding surface. This approach doesn't model the failure mechanism nor disaggregation of the mass after failure, however it gives insight into the influences of the overall slope angle and strength loss. The following paragraphs present the thinking behind the elicited failure velocities for each of the failure volume scenarios.

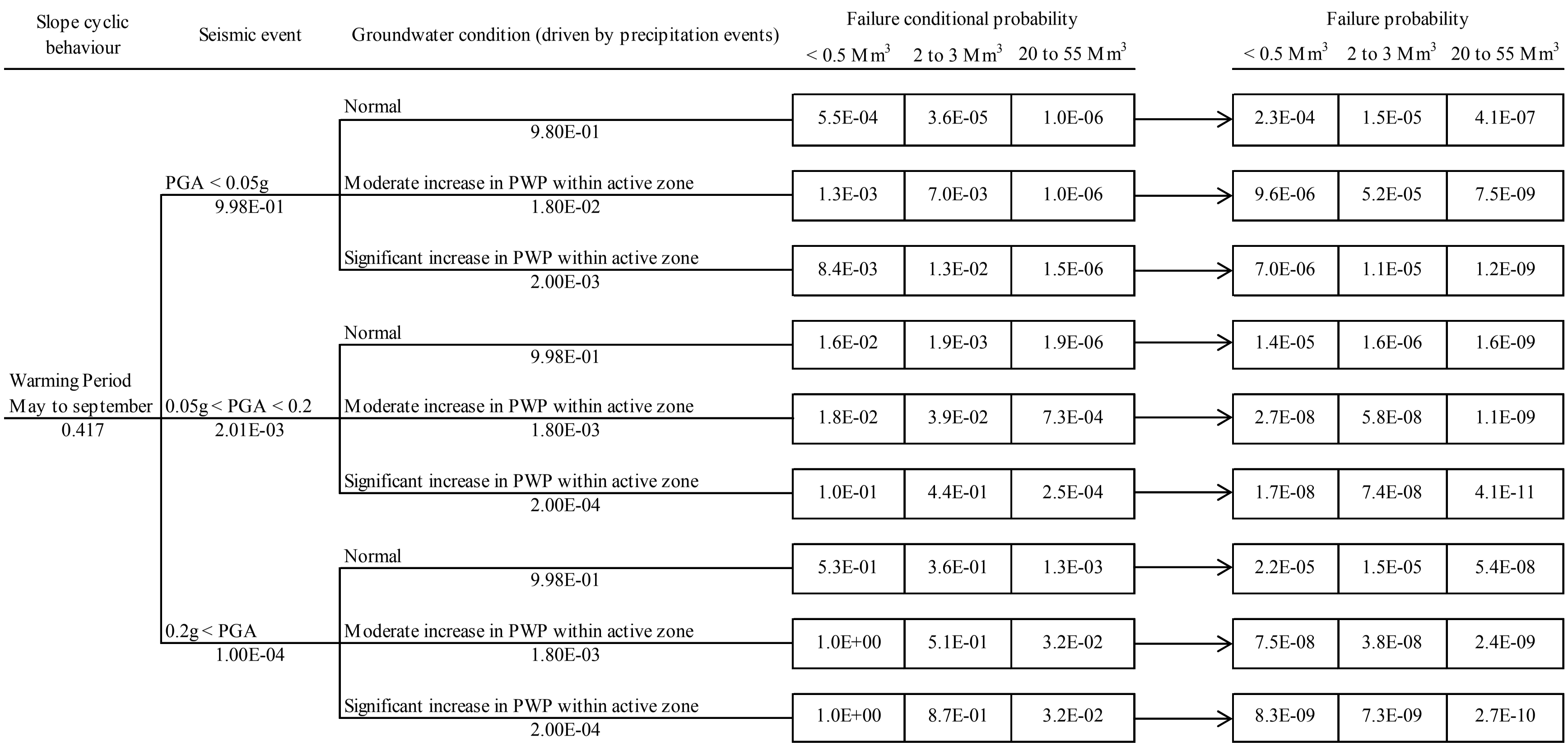


Figure 4-7 Populated event tree for estimating the annual probability of failure for the Checkerboard Creek rock slope - warming period. Values shown correspond to one iteration of the simulation.

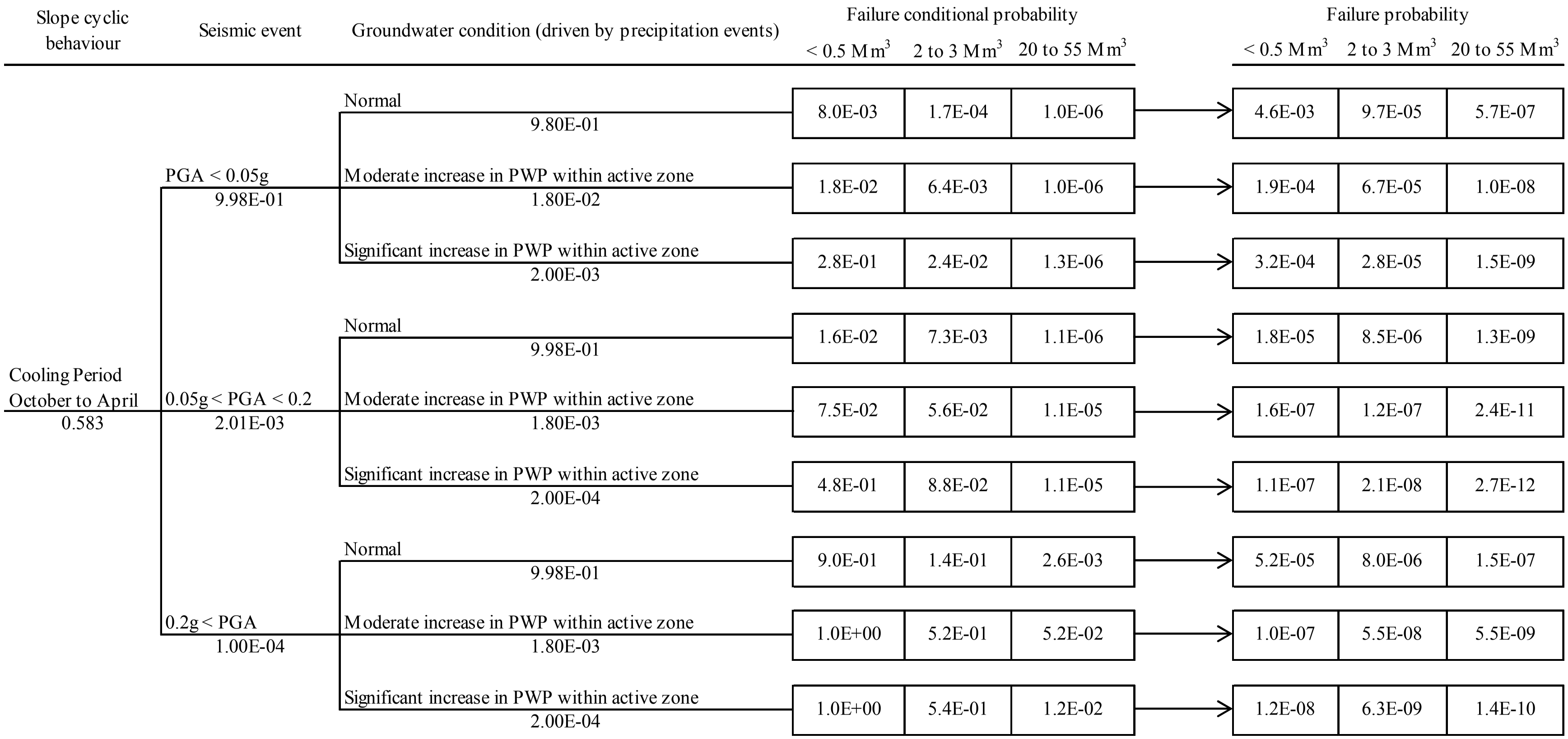


Figure 4-8 Populated event tree for estimating the annual probability of failure for the Checkerboard Creek rock slope - cooling period. Values shown correspond to one iteration of the simulation.

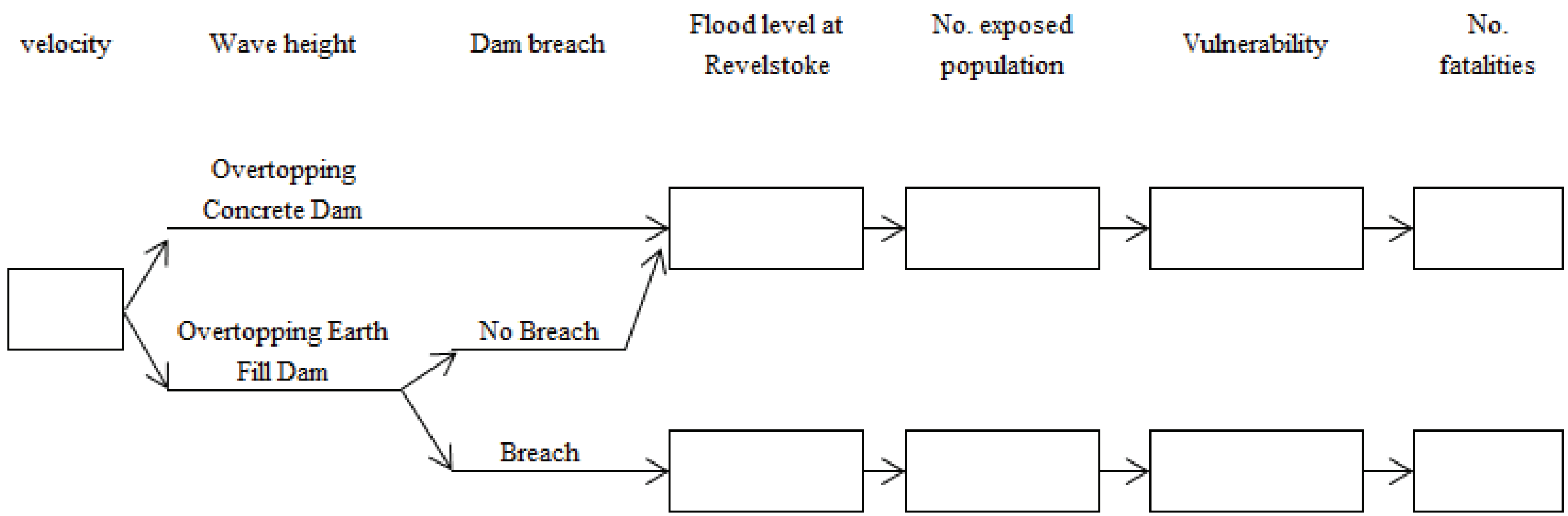


Figure 4-9 Process model developed to estimate the likelihood and number of fatalities given a sudden surge is generated after failure of the Checkerboard Creek slope and dam overtopping or breach occurs.

a) < 0.5 Mm^3: This volume is representative of the most active area of deformation, which corresponds to the steep slope cut above the highway. Simple sliding block calculations and run out analyses using a distinct element code have been developed for this scenario (Watson *et al*. 2006, Lorig *et al*. 2009). Collapse of the slope was controlled by downgrading the tensile strength of the intact blocks. This simulates the mass weathering and disaggregation processes. The velocity of this volume entering the slope was found to be bounded between 20 and 40 m/s. The models indicate that a significant portion of the failed volume is constrained and slowed down by the presence of the highway. However, it was decided to adopt this velocity range for volumes up to 0.5 Mm^3.

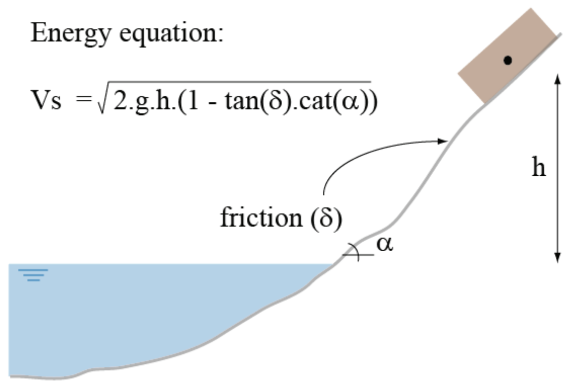


Figure 4-10 Energy equation adopted to assess a rock slide bounding velocities (Vs) when entering a water body.

b) 2 to 3 Mm^3: This volume corresponds to the entire deforming area. Monitoring has shown that the deformation mechanism is neither sliding along a basal shear zone, nor flexural or block toppling, but discrete blocks sliding and rotating relative to each other. Progressive loss of strength could potentially lead to rapid collapse of the mass. However the amount of deformation required for this mechanism to lead to such strength losses could result in the disaggregated mass running downslope at a wide range of velocities. This progressive failure could result in ductile behaviour of the failing mass or lead to brittle failure after a triggering event. It was decided to consider velocities in the upper range similar to those obtained by the smaller volumes but also include the possibility of lower velocities when entering the reservoir. A sliding block model was analysed considering a height of the mass between 40 and 80 m, sliding at angles between 20 and 30 degrees (consistent with the slope geometry and simulated volumes). Frictional resistance along the sliding surface varied between the typical friction angles of rock fill at low stresses (50 degrees after Barton 2008) and less than half the base friction angle of intrusive rocks (15 degrees). The adopted range between 5 cm/s and 40 m/s was given a triangular probability distribution towards the higher values as shown in Figure 4-11.

c) 20 to 55 Mm^3: Geological investigations and observed slope behaviour (Watson *et al*. 2004, Watson *et al*. 2006) do not indicate the existence or formation of a basal shear surface that would allow a volume of 20 to 55 Mm^3 to slide towards the reservoir. The rock beneath the active deformation zone is generally fair to good in quality, and discontinuities dip into the slope. A failure of these volumes would be associated with toppling and/or mass disaggregation mechanisms which are expected to show slow velocities on the initial stages of slope deformation, allowing for updating of the analysis presented in this chapter. The sliding block model considered a height of the mass centre of gravity between 120 and 200 m above the reservoir, The sliding surface inclination varied between 5 and 45 degrees. The frictional resistance was varied between 50 and 15 degrees. The uncertainty in this scenario is expressed by adopting uniform probabilities for the mass velocity when entering the reservoir between 0.5 mm/s and 55 m/s (Rapid, Very rapid and Extremely rapid according to Cruden and Varnes 1996).

The PDFs adopted for the velocity of the failed mass when entering the reservoir are presented in Figure 4-11. Note that velocities below rapid are neglected. This is consistent with the definition of failure for the Checkerboard Creek slope discussed earlier, which disregards slower deformation velocities given our abilities to successfully deal with them. Also note the first two velocity classes in Cruden and Varnes (1996) are not included in Figure 4-11.

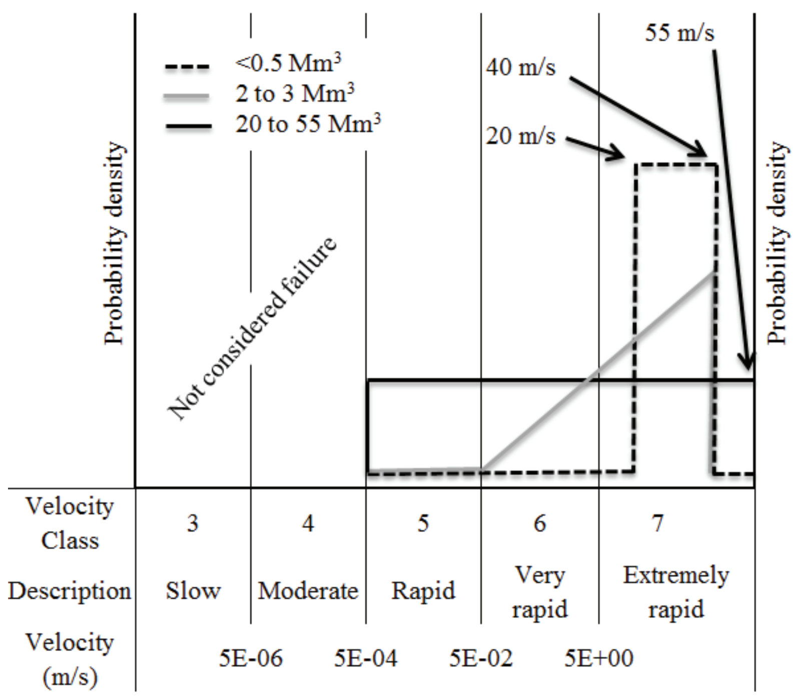


Figure 4-11 Probability density functions adopted for the velocity of the Checkerboard Creek rock slope failed mass when entering the reservoir.

4.3.2.2. Wave Height and Energy

The analysis of impulse wave overtopping heights was performed following the method outlined by Heller *et al*. (2009). The method is classified within the group of generally applicable equations. This means that expressions used to describe the impulse wave generation, propagation and dam overtopping after a landslide enters the reservoir are based on scale models in channels (2D) and rectangular basins (3D). This method allows for sensitivity studies to be performed in short periods of time including variations on water depths, landslide volumes and velocities, dam geometry and freeboard. The outcome is considered a crude estimation given the nature of the idealized geometrical conditions for which the expressions are derived. The uncertainty in the results increases as the real geometry deviates from the ideal models.

The method allows for estimation of the impulse wave characteristics (type, velocity, height) after the wave is generated, it propagates along the reservoir and runs up the dam or shore. It neglects the effect of reflective waves, which increases the uncertainty in the results.

Impulse wave parameters and run up properties were estimated for five locations along the Revelstoke Dam (Figure 4-12). The distance and angle with respect to the slope's dip direction is shown in Table 4-8.

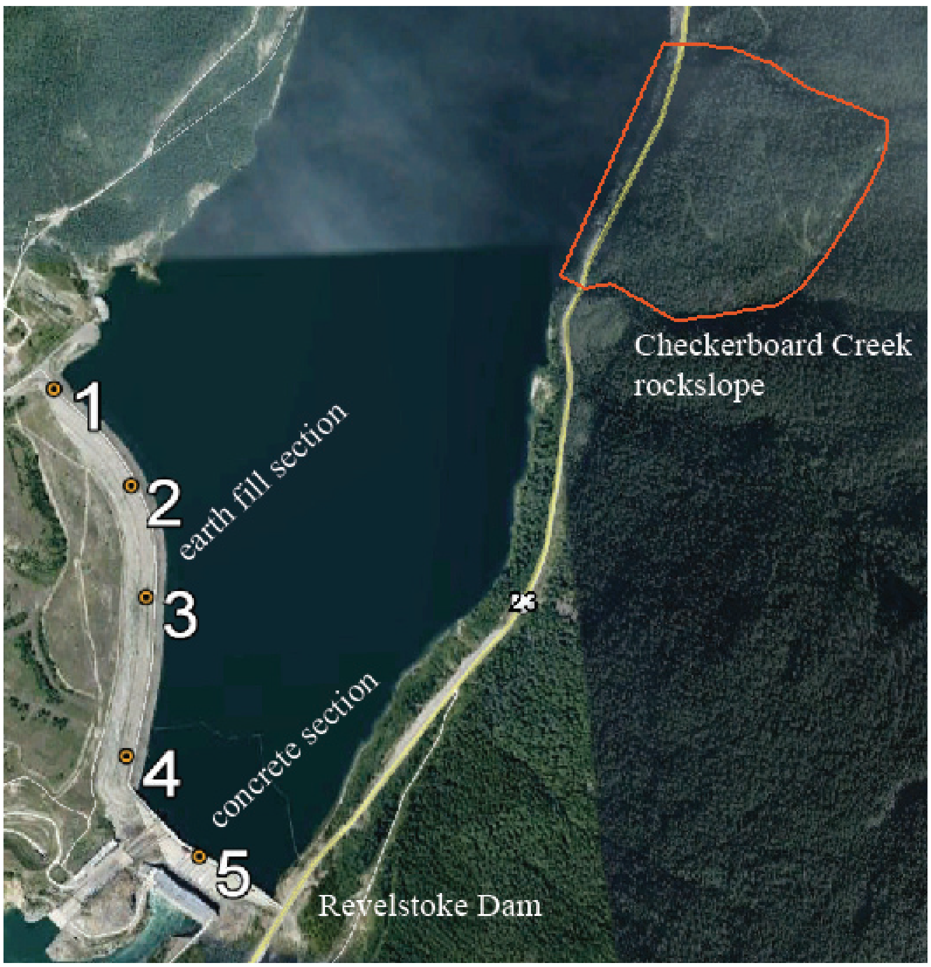


Figure 4-12 Revelstoke Dam and Checkerboard Creek rock slope with locations where the impulse wave run up characteristics were estimated. Ground image extracted from Google Inc. (2012).

Location	Distance (m)	Angle (degrees)
1	1 600	40
2	1 500	55
3	1 770	65
4	2 050	70
5	2 100	78

Table 4-8 Distance between the Checkerboard Creek rock slope and the indicated locations along the Revelstoke Dam (Figure 4-12), and angle relative to the slope's dip direction.

A physical hydraulic wave model was previously developed to assess the overtopping potential after different failure scenarios of the Checkerboard Creek rock slope (Watson *et al*. 2006, Lorig *et al*. 2009). It was reported in these studies that the largest failure volume modelled (1.2 Mm^3) entering the reservoir at the highest speeds considered (between 20 and 40 m/s), caused less than 1 m of overtopping along 200 meters of the earth fill dam. This same scenario caused up to 38 m of run up above reservoir level directly across from the slide. These results were used to validate the method before the sensitivity analyses, and thus reduce the uncertainty in the results. Considering the failing 1.2 Mm^3 to have an average thickness between 20 and 40 m, a width between 100 and 200 m and falling at angles between 40 and 50 degrees, the impulse wave run up heights show to be in agreement with the physical model results published for water depths of 40 m at the slope location and water depths between 40 and 80 m at the dam location. The bulk slide density and porosity were taken as 1700 kg/m^3 and 35% respectively, although results were not particularly sensitive to variations of these parameters.

The earth fill dam was designed to maintain a freeboard of 8 m above the probable maximum flood level (Taylor and Lou 1983). It was assumed the operational level to be about 4 meters below this maximum level, thus the freeboard was assumed to be 12 m. The uncertainty in this assumption and its seasonal variability were taken under consideration when evaluating the run up heights over the dam crest. The freeboard at the concrete gravity dam section is taken as 8 m, the dam crest as 15 m wide for the earth fill dam, and 20 m wide for the concrete gravity dam. The run up angle in the model was 22 degrees for the earth fill dam and vertical for the concrete section.

Table 4-9 presents the range of parameters used in the impulse wave sensitivity analysis. The failure mass geometry ranges are based on the geometric and geological characteristics of the deforming mass and the displacement pattern shown. The mass displacement angle when entering the reservoir was fixed as it showed to have limited influence in the results for angles between 20 and 45 degrees, which were considered representative given the slope characteristics.

Scenario	< 0.5 Mm^3	2 to 3 Mm^3	20 to 55 Mm^3
Slide volume	0.5 Mm^3	2 - 3 Mm^3	20 to 55 Mm^3
Slide thickness (m)	10 - 20	30 - 50	50 - 80
Slide width (m)	50 - 100	150 - 250	600
Slide velocity (m/s)	20 - 40	5 - 40	10 - 55
Slide displacement angle (degrees)	45	30	20

Table 4-9 Range of parameters used in the impulse wave sensitivity analysis - Checkerboard Creek study.

Overtopping of the concrete gravity dam (point 5 in Figure 4-12) was only observed for a failure of the entire slope (20 to 55 Mm^3) sliding at the highest velocities. Under this scenario, overtopping was estimated at about 7 to 10 m. No other volume scenario or slide velocity resulted in significant overtopping of the concrete dam. Figure 4-13, Figure 4-14 and Figure 4-15 present the overtopping heights along the earth fill dam for the different scenarios analysed. Figure 4-13 shows there is no significant overtopping expected for a failure volume less than 0.5 Mm^3.

The methodology proposed by Heller *et al*. (2009) for analyzing the impulse wave run up characteristics includes estimation of the overtopping volume, discharge, and forces acting against the dam. The overtopping discharge is estimated assuming no freeboard, and is taken as an upper limit of the potential discharge. The maximum discharge is estimated as twice the average discharge. In this study, the overtopping discharge for the assumed freeboard was approximated with the simple relation:

$$Q_{f=12} = Q_{f=0} \, (V_{f=12} / V_{f=0})$$

where:

> $Q_{f=12}$ and $Q_{f=0}$ are the overtopping discharge in m^3/s per metre dam crest length, for a freeboard of 12 and 0 m respectively, and;
>
> $V_{f=12}$ and $V_{f=0}$ are the overtopping volume in m3 per metre dam crest length, for a freeboard of 12 and 0 m respectively.

Figure 4-16 shows the average and maximum overtopping discharge as a function of the overtopping height according to the simulations performed. The expressions adopted to estimate the overtopping discharges are based on 2D investigations and neglect the effects of dam curvature and asymmetrical wave impact. Heller *et al*. (2009) discusses a qualitative approach to estimate the influence of this simplification.

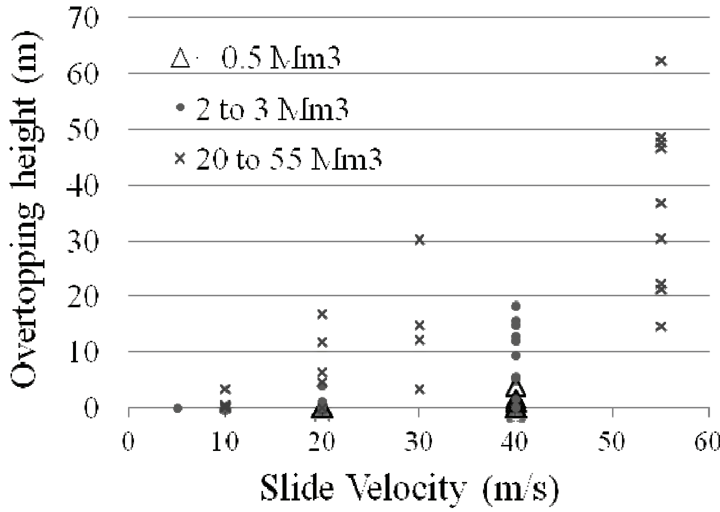


Figure 4-13 Estimated overtopping heights for different failure volume scenarios and slide velocities - Checkerboard Creek study.

Figure 4-17 shows the horizontal and vertical forces applied to the earth fill dam as estimated following Heller *et al*. (2009). The force applied to the dam by the impulse wave is modelled as an hydrostatic force, dependent of the run up height, and showing a triangular distribution which increases towards the bottom. Predictions of these forces are limited by great uncertainty (Heller *et al*. 2009) and values should be taken as order of magnitudes in preliminary assessments.

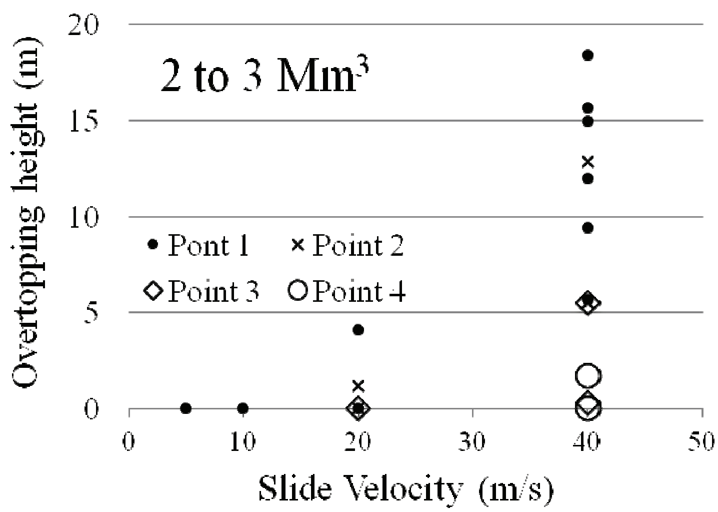


Figure 4-14 Estimated overtopping heights for different slide velocities at different points along the earth fill dam (as per Figure 4-12) - 2 to 3 Mm^3 scenario - Checkerboard Creek study.

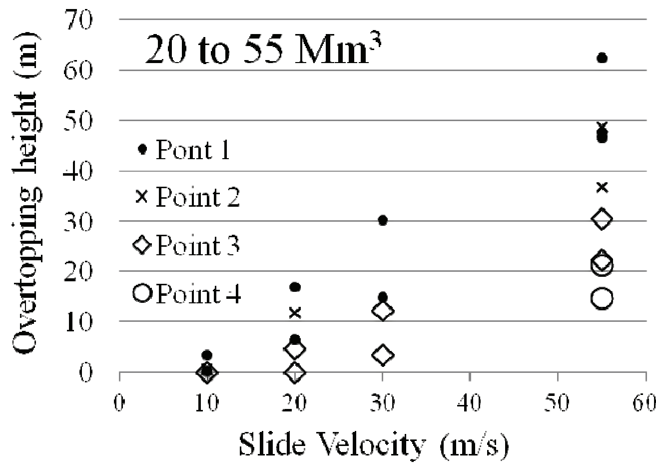


Figure 4-15 Estimated overtopping heights for different slide velocities at different points along the earth fill dam (as per Figure 4-12) - 20 to 55 Mm^3 scenario - Checkerboard Creek study.

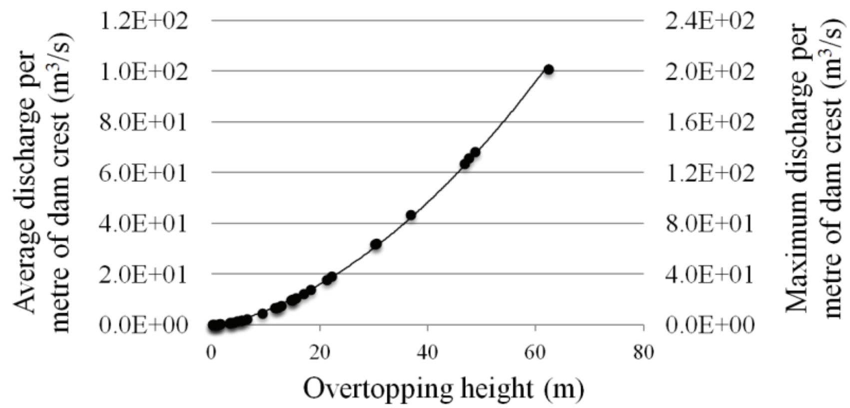


Figure 4-16 Average and maximum discharge per metre dam crest length - Checkerboard Creek study.

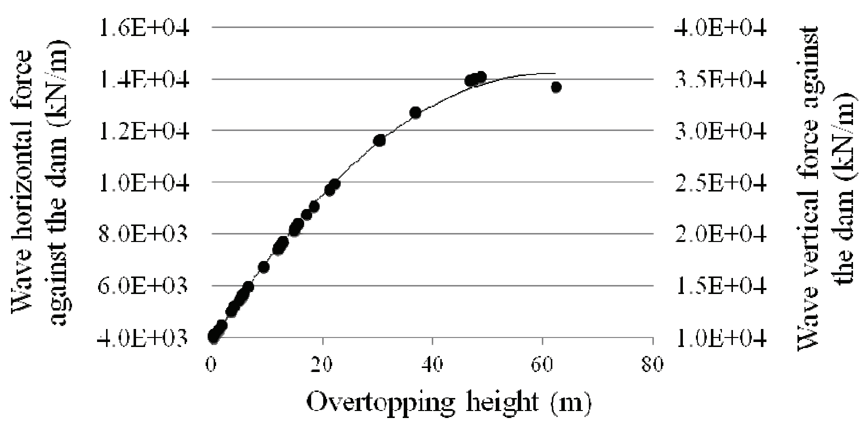


Figure 4-17 Wave horizontal and vertical force against the earth fill dam, per metre dam crest length - Checkerboard Creek study.

4.3.2.3. Earth Fill Dam Robustness Against Overtopping

Unlike wind generated waves, impulse waves are not periodical (only one initial wave and few reflections are expected) but can show significantly higher wave celerity and overtopping height and discharge. Models to analyse dam breach caused by wind generated waves have been developed (Wang and Bowles 2006, Shewbridge *et al*. 2010), however, there are no fully developed models focused on earth fill dam robustness against overtopping of large impulse waves. Balmforth *et al*. (2008) suggested some moraine dam failures can be attributed to dam breaching following overtopping of large impulse waves. In this regard, they tested dam physical models of granular material under impulse waves and presented a theoretical model to rationalize their observed results. This model assumed the dam material erosion rate to be proportional to the square of the flow velocity.

Figure 4-18 shows a simplified sketch of an impulse wave overtopping an earth fill dam. The potential effects of the overtopping wave against the dam are also shown (erosion, impact forces and hydraulic forces). In this study the robustness of the Revelstoke earth fill dam against impulse wave overtopping is assessed based on two potential failure mechanisms: erosion of the embankment materials and embankment instability caused by the impact and hydraulic forces.

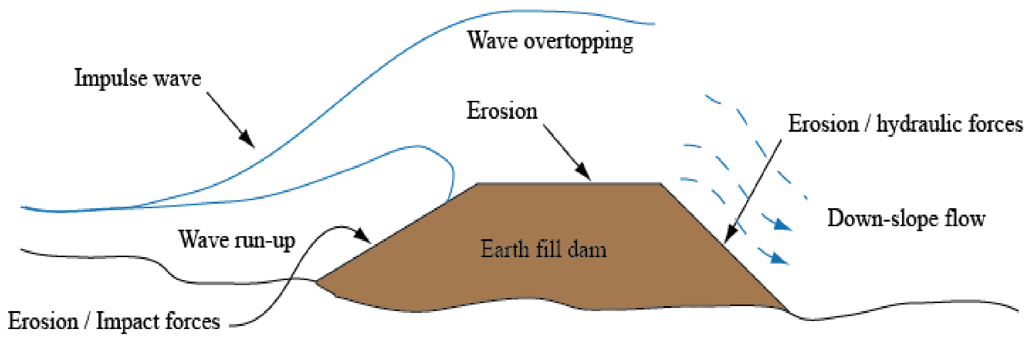


Figure 4-18 Simplified sketch of impulse wave overtopping an earth fill dam.

Erosion

A simple model relating erosion rate to hydraulic and geotechnical parameters is presented in Shewbridge *et al*. (2010). The model relates the erosion rate to the soil shear strength and erodibility, as well as the flow effective hydraulic stress applied:

$$e = k\,(t - tc)$$

where:

e is the erosion rate,

k is the erodibility coefficient,

t is the hydraulic stress, and;

tc is the critical shear stress of the material conforming the dam.

The critical shear stress of the materials forming the dam (tc) and the erodibility coefficient (k) can be taken as 4 psf (0.2 kPa) and 0.01 ft^3/lb-hr (0.0006 m^3/kg-hr) respectively. These correspond to very resistant material (Shewbridge *et al*. 2010) which is consistent with the Revelstoke earthfill dam characteristics.

The hydraulic stress can be estimated as:

$$t = 0.5\ p\ f\ u^2$$

where:

p is the density of water,

f is the friction factor, and;

u is the current speed or water velocity at the contact with the dam.

Puleo and Holland (2001) and Shewbridge *et al*. (2010) discuss methods to estimate the friction factor (f). Depending on the hydraulic characteristics and the slope roughness, this parameter is highly variable. In this study the friction factor was varied between 0.05 and 0.005.

Different erosion rates will be associated for wave run up velocities, flow velocities on the dam crest and at the downstream dam slope. Studies assessing flow velocities during sea dike overtopping (Schuttrümpf and Van Gent 2003, Pullen *et al*. 2007) are limited to relatively small overtopping heights and small amplitude waves (such as wind generated waves), when compared to large landslide-generated-impulse-waves. Estimating impulse wave velocities with approaches developed for different flow and overtopping conditions is associated with great uncertainty.

Numerical approaches can be used to simulate impulse wave generation and propagation (Falappi and Gallati 2007, Quecedo *et al*. 2004, Zweifeld *et al*. 2007). These can also be used to estimate the flow characteristics during dam overtopping, however they require much effort, are time consuming, and require some calibration to reduce uncertainty in the results. This is considered a next step in case the erosion process be considered as potentially critical for dam stability after impulse wave overtopping.

An initial assessment was carried out for this study considering a tentatively extreme event (maximum overtopping heights) and the methodology and material properties described above. It was found that even when high flow velocities are associated with high erosion rates, the short duration of overtopping leads to limited erosion of the structure. The wave generation and propagation analysis indicated impulse waves having a celerity of about 20 m/s when reaching the earthfill dam. To illustrate the significance of the short duration of the overtopping wave, flow velocities one order of magnitude higher than the wave celerity (about 200 m/s) would be required to induce a breach of the earthfill dam. The scenario is then considered extremely unlikely and no further analysis required for this study.

Structure Stability - Wave Impact

During dam overtopping, the impulse wave will apply a pressure on the upstream slope of the dam. This pressure cannot be considered as static given the short duration of overtopping. It cannot be considered as an impact load as it increases during run up and overtopping, reach a maximum value, and then decreases as the water level lowers again. During the short duration overtopping it is expected that inertial forces aid the dam increasing its resistance to such event. Moreover, neglecting strain rate effects, a limit equilibrium analysis should provide a conservative assessment of the dam overall stability condition under large overtopping scenarios.

Figure 4-17 showed the horizontal and vertical wave-induced forces against the dam. According to Heller *et al*. (2009) this force is distributed as a pressure along the slope height. The stability analysis considered the maximum forces applied given the highest overtopping event occurs. The force was then assumed to be distributed along the upper half of the slope. This was modelled as a 400 kPa surcharge load normal to the slope surface.

Details on the dam section and design, material types, and construction procedures were taken from Taylor and Lou (1983) and Salmon (1988). The analysis was performed with the software SLOPE/W part of the GeoStudio suit (GEO-SLOPE 2007). The method of analysis chosen was the Morgenstern-Price method. A first analysis was performed assuming the core material is un-drained. The dam showed significant robustness against the applied loads. a second analysis was done assuming the core material to show a drained behaviour.

The main analysis considerations were:

- The analysis considered only one section of the dam,
- The reservoir is at maximum design level. This implies maximum hydrostatic load and pore pressures within the dam body,
- It was assumed the piezometric elevation within the dam, upstream of the core section, is that of the reservoir level. The piezometric elevation then reduces linearly to the toe of the exposed downstream slope. This is considered an extreme condition (conservative) given the characteristics of the dam section,
- The wave load is considered as an un-drained load. Materials were given a value of B-bar to simulate an increase in pore pressures due to the wave loading, and;
- Inertial forces and strain rate effects were not considered.

Figure 4-19 and Figure 4-20 show the limit equilibrium analyses. Material parameters and values of B-bar are also shown. The analysis suggest that even under a conservative loading scenario, the earthfill dam has strength reserves against overall failure.

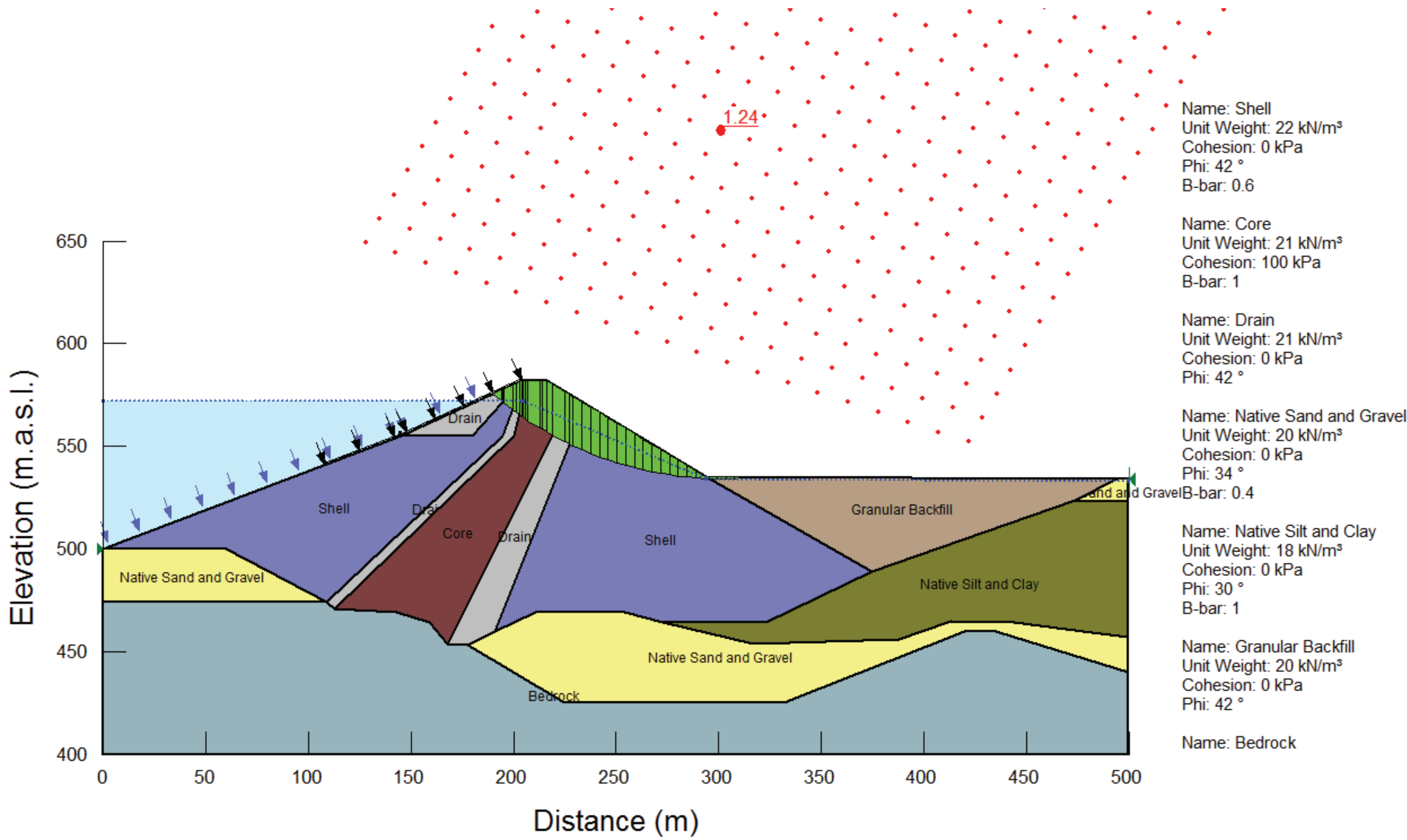


Figure 4-19 Revelstoke earthfill dam stability analysis for worse case overtopping wave expected given a failure of the Checkerboard Creek slope occurs (pressure of 400 kPa on the upper half of the upstream slope). The core material has un-drained behavior. Inertial forces and strain rates during the short overtopping period are neglected. Material strength parameters considered similar to those used for design.

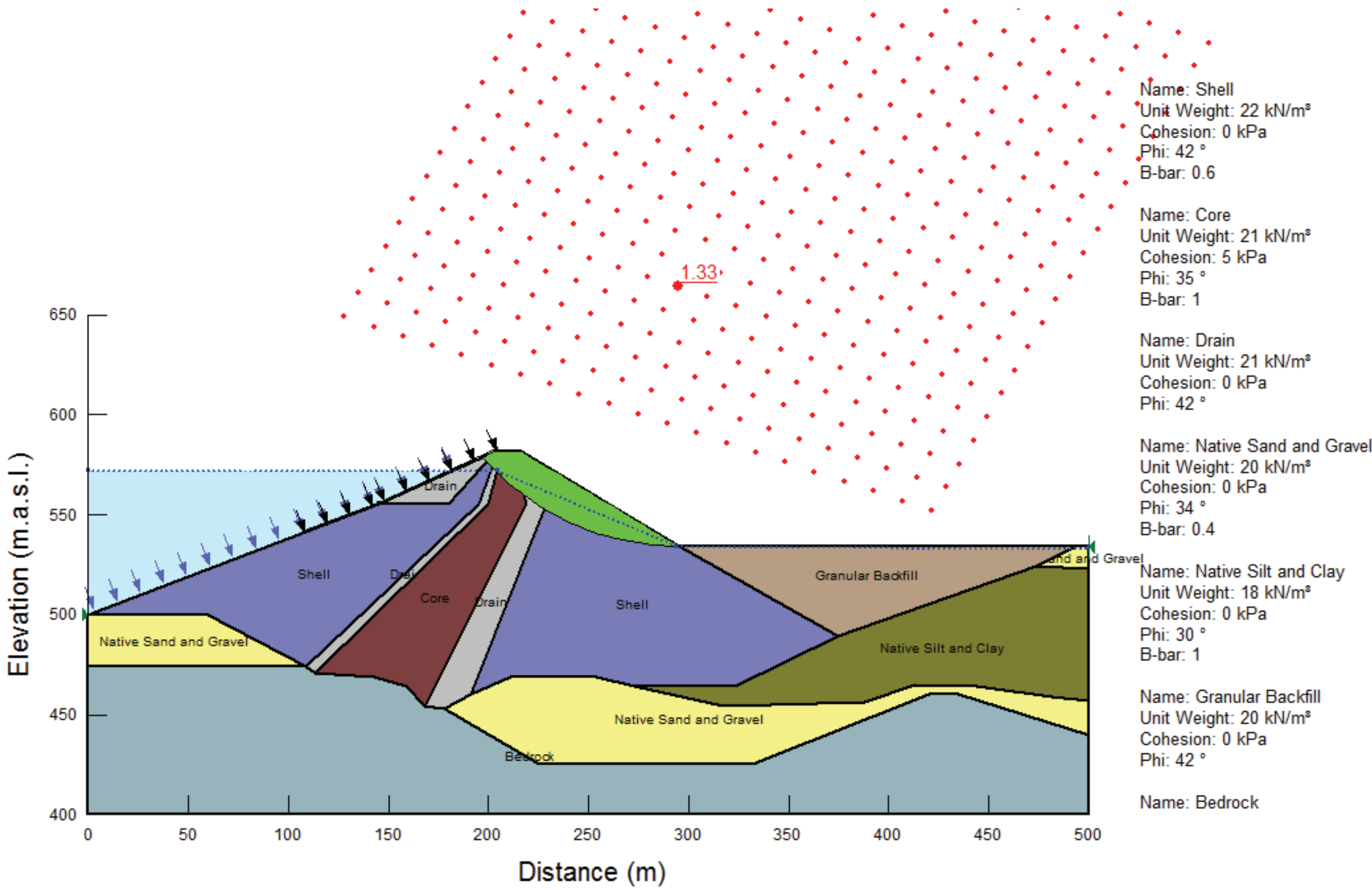


Figure 4-20 Revelstoke earthfill dam stability analysis for worse case overtopping wave expected given a failure of the Checkerboard Creek slope occurs (pressure of 400 kPa on the upper half of the upstream slope). The core material has drained behavior. Inertial forces and strain rates during the short overtopping period are neglected. Material strength parameters considered similar to those used for design.

Structure Stability - Downslope Flow

The downslope flow after an impulse wave overtopping will apply a shear force along the slope face, thus reducing the stability of the structure. There are, however, significant uncertainties in the estimation of this downslope flow characteristics after large impulse-wave overtopping. Figure 4-21 presents an idealized downslope flow profile after impulse-wave overtopping. This assumes that the critical situation can be described by linear water surface between the toe of the slope, where the amount of flow is zero, and the head of the slope, where the water height it's at is maximum. It is recognized this is an oversimplification and that further research on large impulse wave overtopping flow is needed.

The maximum height at the crest of the downstream slope (h max) can be approximated as 0.41 the overtopping height (after Pullen *et al*. 2007). The flow discharge (q) is taken as the maximum from Figure 4-16 (most conservative).

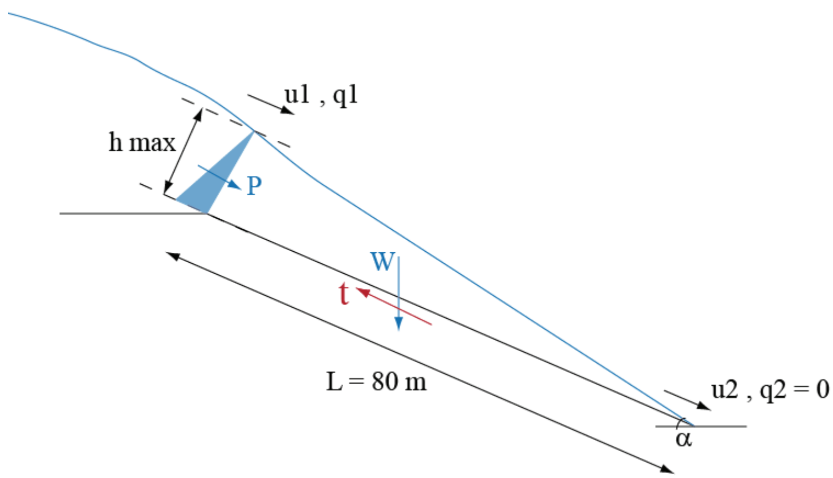


Figure 4-21 Idealized downslope flow after impulse-wave overtopping.

The following expression was used to estimate the shear force applied to the downstream slope face (after Tingsanchali and Chinnarasri 2001):

$$t = P + (\rho\, q1\ u1) + W \sin(\alpha)$$

where:

- P is the hydrostatic water force,
- ρ is the density of water,
- q1 and u1 are the flow discharge and velocity, respectively,
- W is the weight of the water wedge, and;
- α is the downstream slope angle.

This approach assumes that the water wedge is in equilibrium, which is not the case. The uncertainties related to this assumption need to be considered when assessing the results from the stability analysis of the slope. In this step of analysis, the material properties were taken similar to those used for design according to Salmon (1988). However, conservative flow velocities and discharges were adopted. Again, the inertial forces and strain rates were not considered which should be kept in mind considering the scenario analysed is of short duration (probably one or two seconds at most of the overall 20 second overtopping duration).

The estimated shear force was distributed along the downstream slope. The stability was evaluated through the limit equilibrium models previously described. The software does not allow for distributed shear stresses to be defined, so this was mimicked by 20 point loads in the direction parallel to the slope (Figure 4-23). The weight of the water was modelled as a surcharge load. Figure 4-22 shows the material parameters and stability analysis of the Revelstoke earthfill dam under downslope flow after 10 m of wave overtopping. Figure 4-24 plots the results for diverse overtopping heights and for the safety factor against failure of the downstream slope and overall dam failure. Note that failure of the downstream slope as shown in Figure 4-22 could lead to imminent overall dam failure.

Given the conservative assumptions adopted (neglecting inertial forces and strain rate effects, as well as the short duration of the event), the results indicate that there could be a limited probability of earthfill dam breach for overtopping heights over 50 m, and increasing as the overtopping heights go above this height.

4.3.2.4. Flood Analysis

The two primary tasks of a flood analysis after a dam breach are the prediction of the reservoir outflow hydrograph and the routing of this flow (Wahl 2010). Models dealing with outflow hydrograph prediction can be grouped in (Gee 2010): Regression Equations (Froehlich 1987, MacDonald and Langridge-Monopolis 1984), Process Models like BREACH (Fread 1988), SIMBA (Hanson *et al*. 2005) and ERODE (Marche 2005), and Federal Agency Guidelines (USACE 1980). The later was adopted to develop a preliminary assessment. The software HEC-RAS Version 4.1.0 (USACE 2010) was used to model the water wave as a transient flow. The model geometry and output visualization was done through HEC-GeoRAS Version 4.2 (USACE 2009), an ArcGIS (ESRI 1999) extension that allows import and export capabilities between HEC-RAS and ArcGIS.

Given a wave overtops the dam, there are two flooding scenarios: 1) The overtopping wave leads to a breach of the earthfill dam, thus a flood wave arriving to Revelstoke, and 2) The overtopping discharge reaches the town, no earthfill dam breach. A preliminary assessment of the potential flooding at Revelstoke under these scenarios was performed for this study. The topography of the area was obtained from NRC (2011) at a scale of 1:50,000. There was no detail of the river bed topography available for the study. To overcome this, a sensitivity analysis was performed for the depth of the river channel considering a steady flow between 3000 and 6000 m^3/s, and for a stream gradient of 0.0001, typical of the river section in the area. Given the uncertainties introduced by this approach, results obtained are taken only as a magnitude of the potential flooding that can be expected for each scenario rather than more accurate predictions.

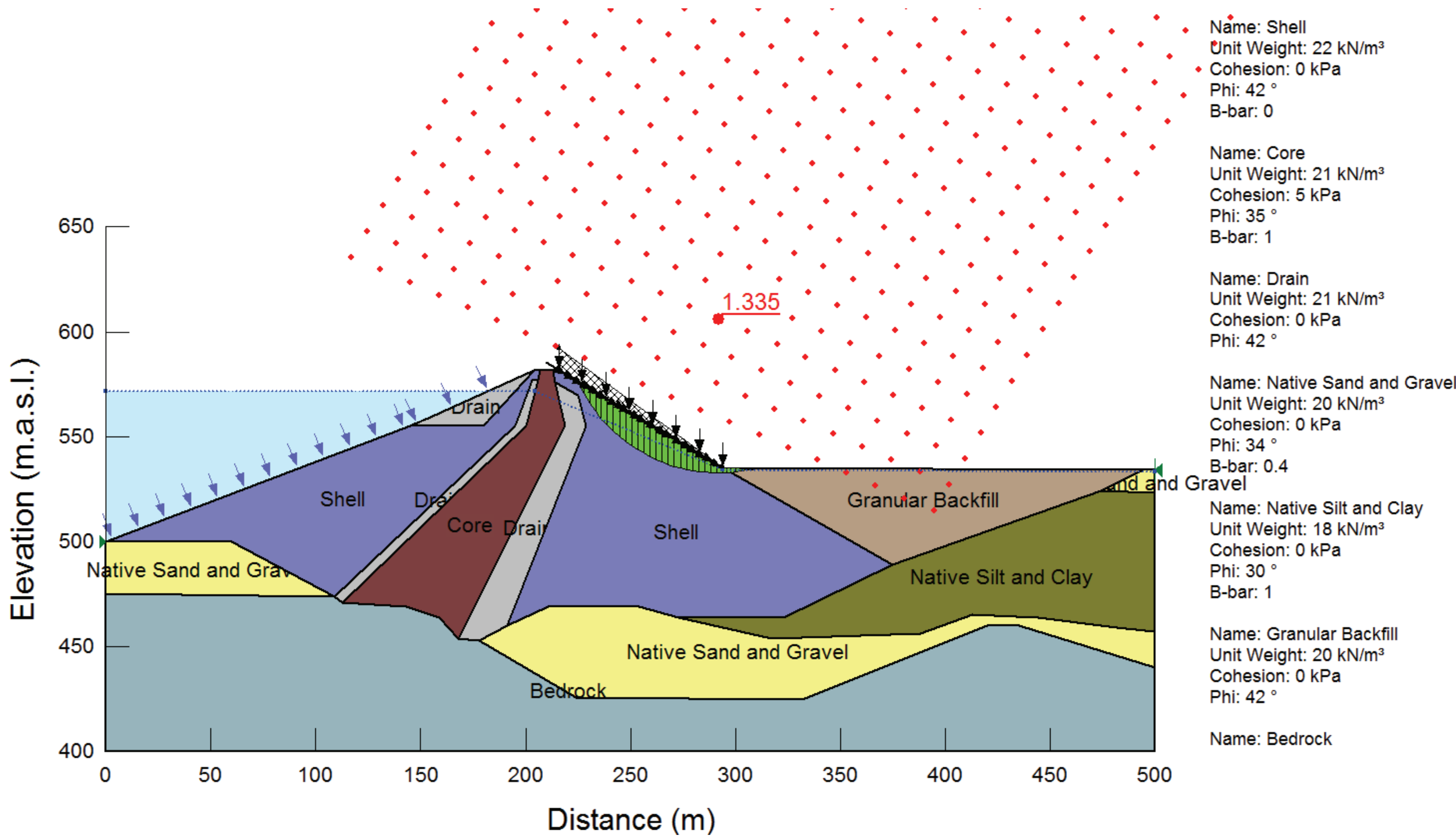


Figure 4-22 Material parameters and stability analysis of the Revelstoke earthfill dam under downslope flow after 10 m of wave overtopping.

It was assumed in the model that the breach develops in a period of 15 minutes. Given the dimensions of the reservoir, it was also assumed that the water level in the reservoir remains constant for another 15 minutes. The breach model was set to develop to the base level of the earthfill dam downstream toe (30 m below the reservoir water level). The two breach scenarios analysed considered a breach base width of 300 m and 600 m corresponding to about 1/4 and 1/2 of the earhfill dam length. This dam breach scenarios are considered conservative.

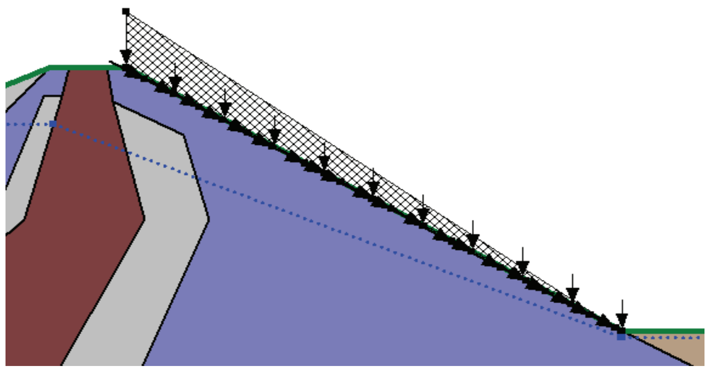


Figure 4-23 Detail of the SLOPE/W model of the Revelstoke earthfill dam showing the mimicked shear load due to downslope water flow after wave overtopping.

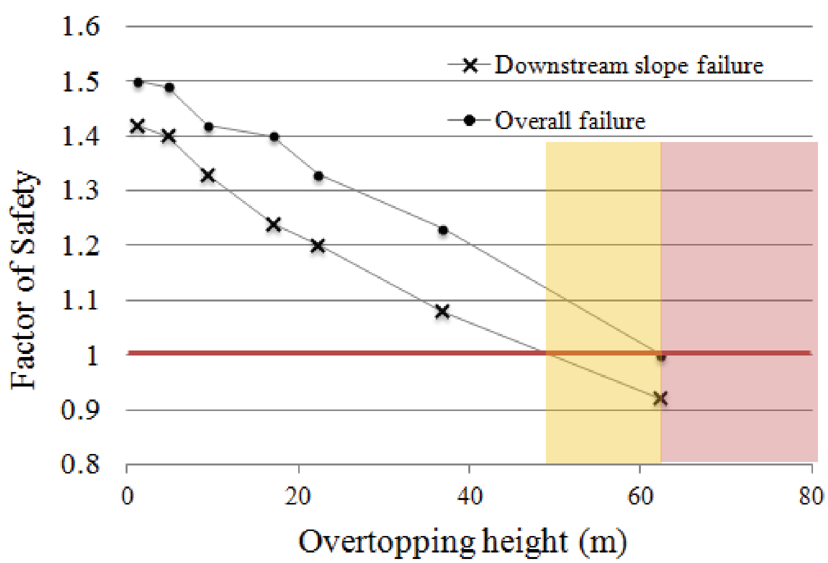


Figure 4-24 Stability analysis results of the Revelstoke earthfill dam for the downslope flow after wave overtopping.

The overtopping waves were also modelled as unsteady flow in HEC-RAS. The wave generation and overtopping analysis indicated overtopping durations between 20 and 30 seconds. The models considered wave durations of 20 minutes in order to avoid numerical instabilities. As such, the estimated floods for the wave overtopping analysis need to be taken as conservative upper values. The discharge modeled for the overtopping waves are in agreement with the wave overtopping analysis presented in this chapter considering the location of each evaluation point along the dam crest (Figure 4-12).

Figure 4-25 and Figure 4-26 present the results of the flood analysis at Revelstoke following an earthfill dam breach and overtopping waves with no dam breach, respectively.

4.3.2.5. Flood Level and Human Vulnerability

Based on the flood analyses, three flooding scenarios were defined, as shown in Table 4-10. The flow levels assigned are compatible with the studies described in Peng and Zhang (2012) relating water depths and velocities with flooding levels, which are based on a variety of case studies around the world. These levels are also compatible with those proposed by Graham (1999) for the US Bureau of Reclamation. The classification of flooding scenarios adopted in this study is also dependent on the area flooded, which is related to the number of exposed population. This number of exposed population corresponds to an estimate based on the areas flooded.

Methods to assess life loss after flooding events fall into two categories (McClelland and Bowles 2002): 1) empirically based (Brown and Graham 1988, DeKay and McClelland 1993, Graham 1999 - all for the US Bureau of Reclamation); and 2) rely on parameters considered theoretically important (the BC Hydro method, under development at the time of McClelland and Bowles publication in 2002). The latest US Bureau of Reclamation method, as described by Graham (1999), together with the latest study described in Peng and Zhang (2012), were used to define the vulnerability ranges for each flood level. Note the number of people exposed does not consider evacuation procedures. The method described in Graham (1999) differentiates between varying warning times and its effectiveness to assign different vulnerabilities (or fatality rates).

Scenario	Flood Level	People Exposed	Vulnerability (or fatality rate)
Overtopping waves with up to 50,000 m^3/s peak discharge	L1 - Low severity	Up to 100	0.01 - 0.05
Overtopping waves between 50,000 and 115,000 m^3/s peak discharge	L2 - Medium severity	Up to 1,000	0.1 - 0.5
Dam Breach	L3 - High severity	Up to 8,000	0.9

Table 4-10 Flooding scenarios considered after the analysis results presented in Figure 4-25 and Figure 4-26.

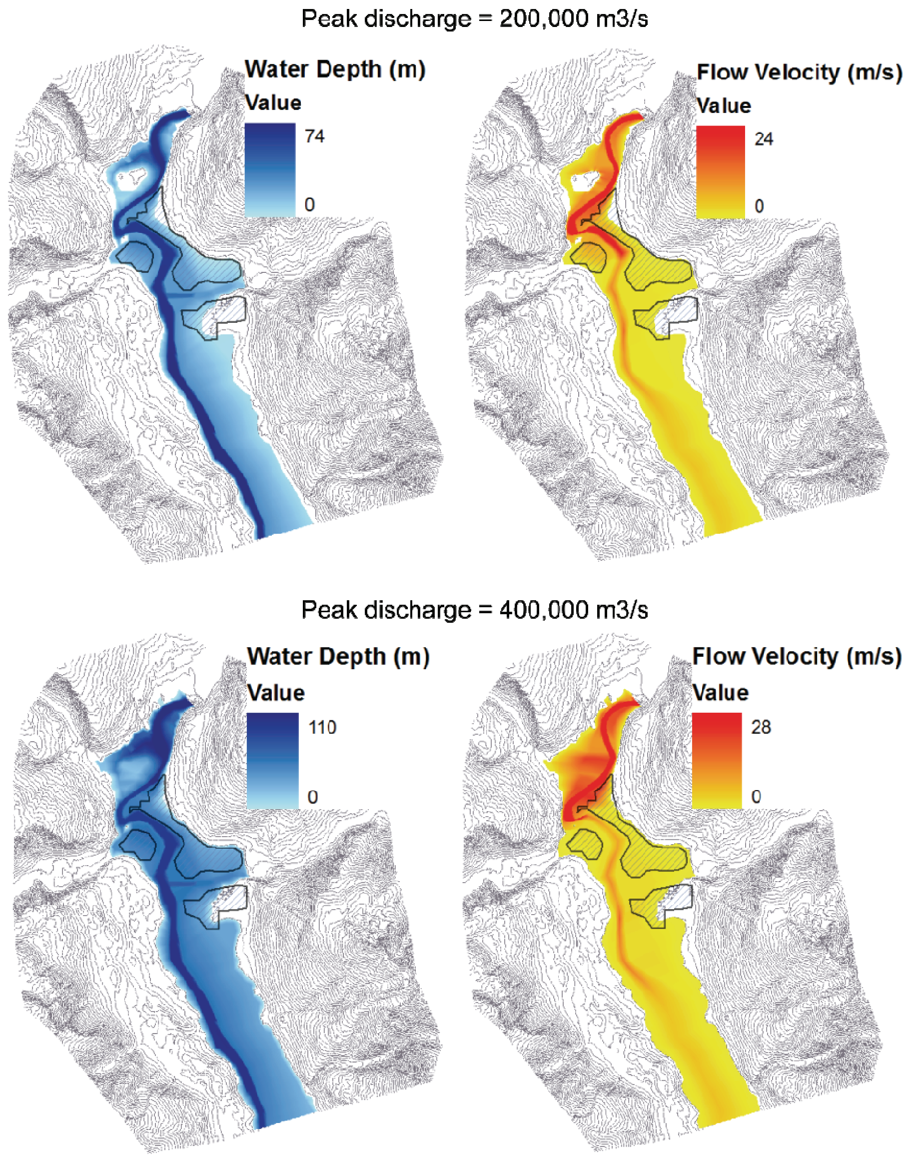


Figure 4-25 Dam breach flood analysis for Revelstoke. Populated areas are delineated.

4.3.2.6. PDFs to Populate the Analysis

Some PDFs need to be defined to populate the Monte Carlo simulations. These are presented here and are based on the analyses previously discussed.

Slide Velocity and Overtopping heights

Given a slope failure occurs, the slide velocity is taken from the PDFs presented in Figure 4-11. The slide velocity is then associated with a PDF for the overtopping height according to the wave generation and propagation results presented in Figure 4-13, Figure 4-14 and Figure 4-15. The ranges of potential overtopping heights as function of slide velocity are presented in Figure 4-27, Figure 4-28 and Figure 4-29 for each of the three volume scenarios. Uniform density functions are assigned within these ranges.

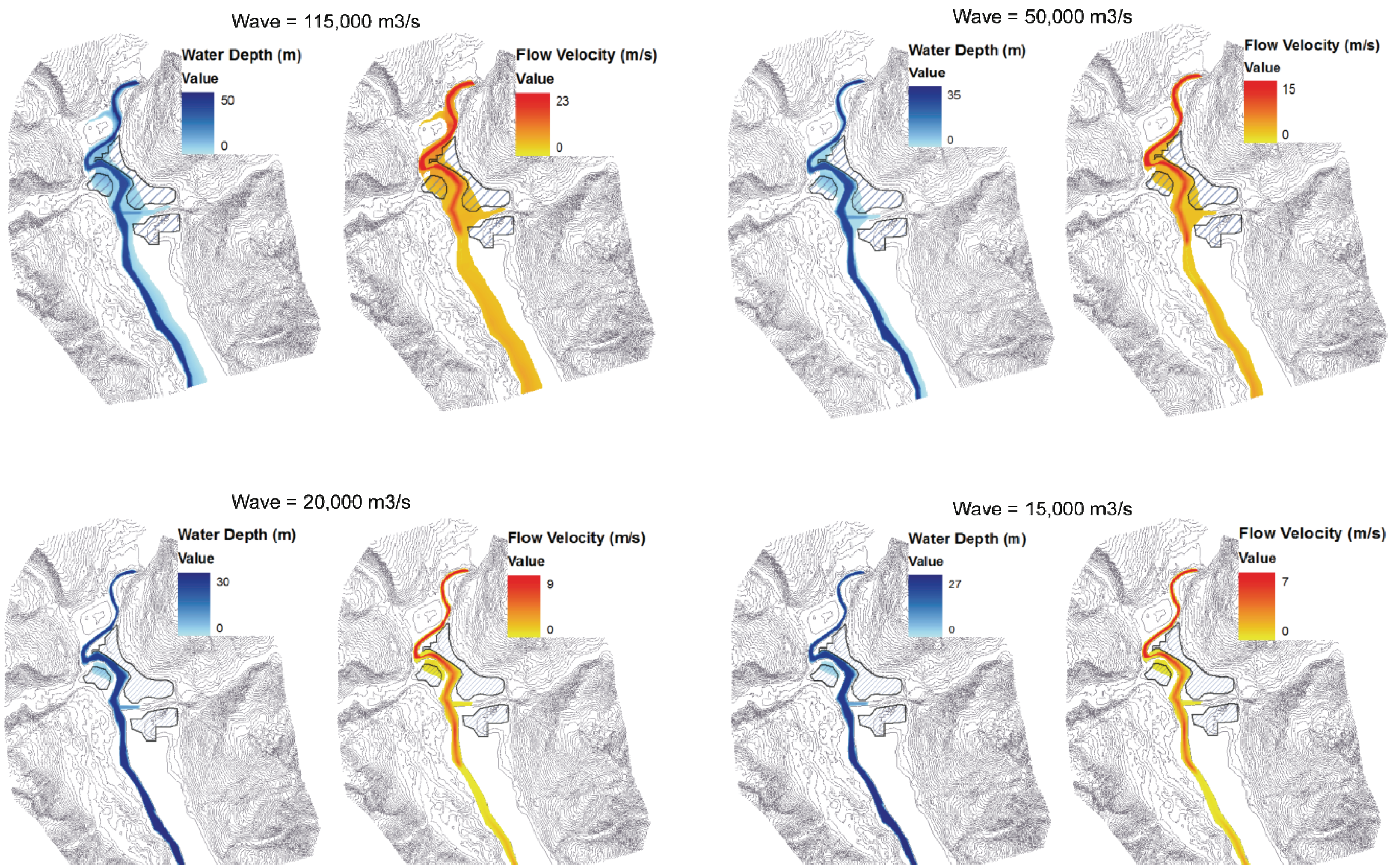


Figure 4-26 Flood analysis at Revelstoke following impulse-wave overtopping. Populated areas are delineated.

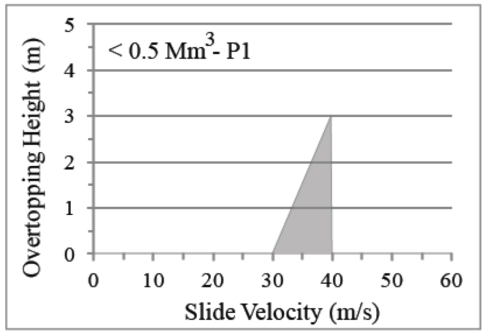


Figure 4-27 Range of potential overtopping heights as function of slide velocity for failure volumes <0.5 Mm^3 - Checkerboard Creek study.

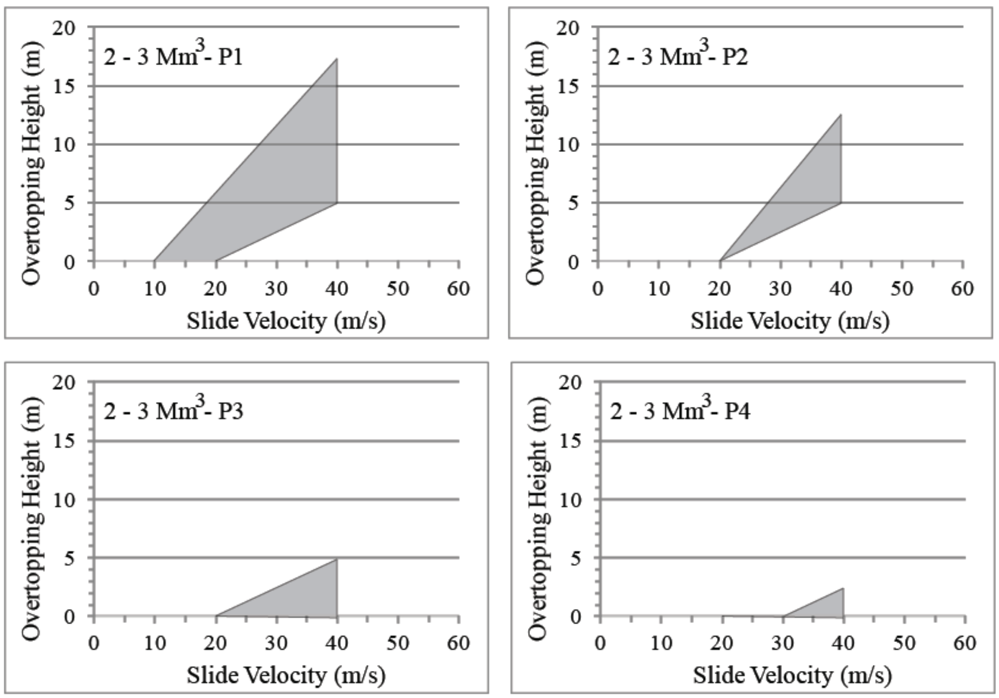


Figure 4-28 Range of potential overtopping heights as function of slide velocity for failure volumes of 2 to 3 Mm^3- Checkerboard Creek study.

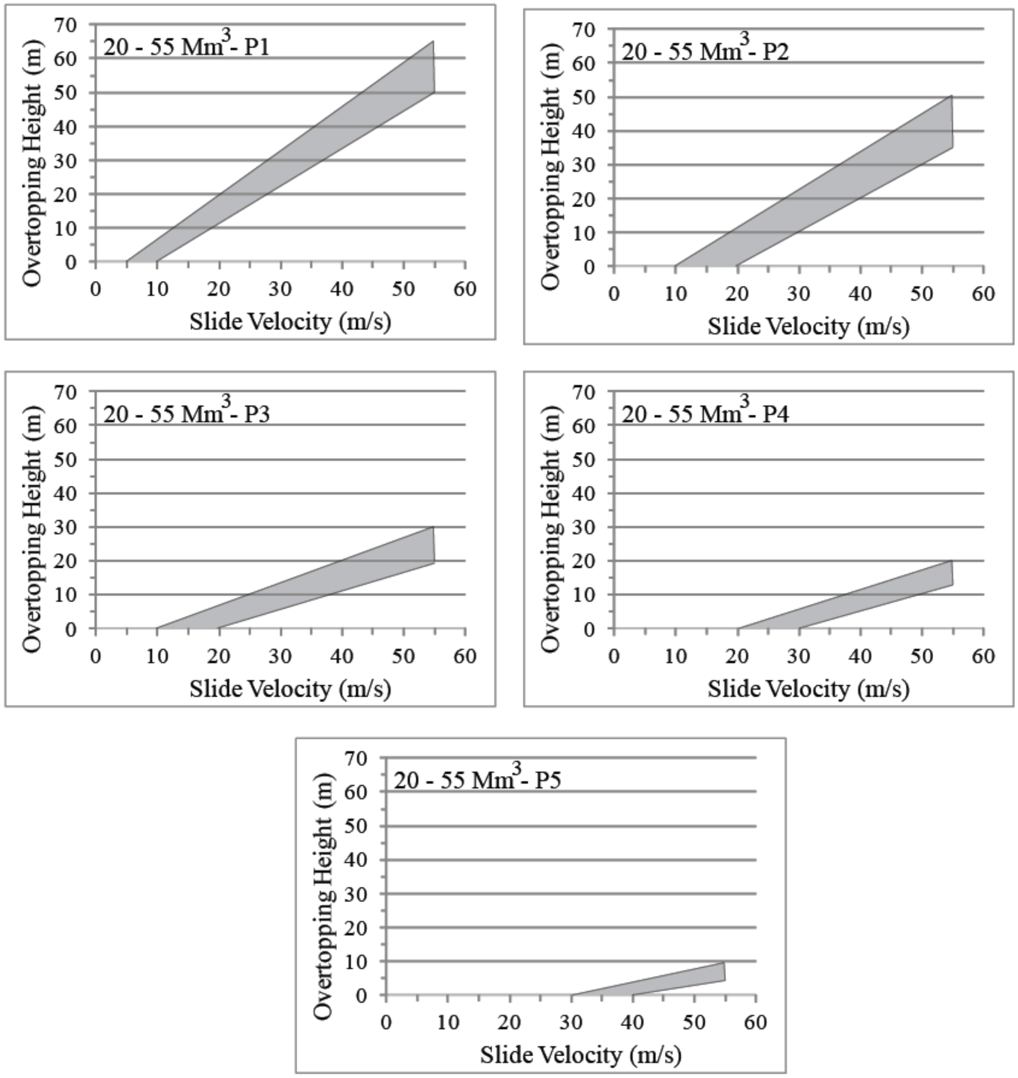


Figure 4-29 Range of potential overtopping heights as function of slide velocity for failure volumes of 20 to 55 Mm^3 - Checkerboard Creek study.

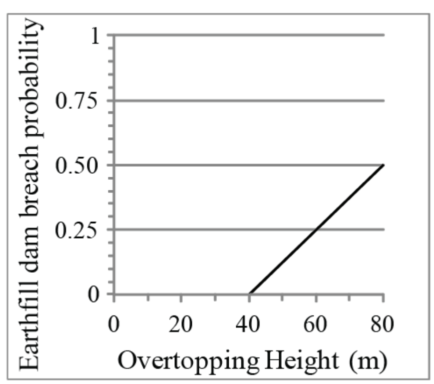


Figure 4-30 Earthfill dam breach upper bound probability as a function of overtopping height - Checkerboard Creek study.

The PDFs were positively correlated with a correlation coefficient of 0.8 to maintain consistency in the relation between slide velocity and overtopping height for the different calculation points.

Flood Level After Wave Overtopping (No Dam Breach)

The overtopping height is then associated with a maximum overtopping discharge per metre of dam crest length (Figure 4-16). An overtopping wave maximum discharge is estimated at each dam calculation point (see Figure 4-12). Each of these calculation points is assumed to represent the average conditions for a certain length of dam crest. Knowing the wave discharge per metre of dam crest and the crest section length for each calculation point, the total discharge can be calculated along the entire dam. The dam crest lengths representative for each calculation point are shown in Figure 4-31 (upper half of the calculations). The estimated total overtopping discharge is then associated with a flood level, following the criterion in Table 4-10.

Earthfill Dam Breach and Flood Level

Based on the analysis on earthfill dam robustness against overtopping, an earthfill dam breach upper bound probability is defined as a function of overtopping height (Figure 4-30). In case breach occurs, the flood level is set to its maximum (Level 3 in Figure 4-31 and Table 4-10) according to the flood analyses presented.

Vulnerability (or Fatality Rate)

The vulnerability in Table 4-10 is presented as a range of values for two of the failure volume scenarios. To populate the Monte Carlo simulation, a uniform probability density function was defined within each range of values.

4.3.2.7. Loss of Life Calculations

A routine to estimate the number of fatalities given a slope failure occurs is presented in Figure 4-31 for the 20 to 55 Mm^3 volume scenario. A similar routine was used for the 2 to 3 Mm^3 volume scenario. The <0.5 Mm^3 volume scenario was not considered given the minimum overtopping expected, if any. A Monte Carlo simulation is then performed over this routine.

The routine starts by randomly selecting a slide velocity according to the defined PDFs. Wave overtopping heights are then assigned for each calculation point following the slide velocity selected. These overtopping heights are then associated with a maximum (or peak) overtopping discharge, and the total wave overtopping discharge along the dam is calculated. A flood level is then assigned for wave overtopping.

Given the maximum overtopping height, the earthfill dam is considered to breach or not, following the breach probability function in Figure 4-30. If no breach results, a flood level of 0 (zero) is assigned for the dam breach flood level. If breach results after overtopping, a flood level of 3 (maximum in Table 4-10) is assigned. The flood level for the entire iteration is then the highest between the wave overtopping flood level and the dam breach flood level.

The exposed population and the fatality rate is then extracted for the flood level according to Table 4-10. The number of fatalities for each simulation is then estimated by the product of the exposed population and their vulnerability (or fatality rate).

Volume (Mm^3)	20 to 55				
Slide velocity (m/s)	48.97				
	P1	P2	P3	P4	P5
O.T. height (m)	57.0	43.2	25.9	16.5	7.5
Crest length (m)	170	330	330	330	475
Peak O.T. discharge (m^3/s)	30,218.59	36,852.07	16,467.78	8,529.96	4,395.94
Total Peak O.T. discharge (m^3/s)	96,464.34				
Flood level due to O.T.	2				
Earthfill dam breach?	Yes				
Dam breach flood level	3		Max Flood level		3
Exposed population	8000				
Fatality rate	0.9				
No. fatalities	7200				

Figure 4-31 A routine to estimate the number of fatalities given a slope failure occurs. Shown is one iteration of the Monte Carlo simulation for the 20 to 55 Mm^3 volume scenario - Checkerboard Creek study.

Each Monte Carlo simulation consisted on 10,000 iterations. This was considered a large statistical sample given the simplicity of the routine. The results were then plotted as number of fatalities (N) versus cumulative probability (F) of N or more fatalities (Figure 4-32). Note that F in this figure corresponds to conditional probabilities given the failure scenario is realized; it is a measure of the consequence given a slope failure, not a measure of risk.

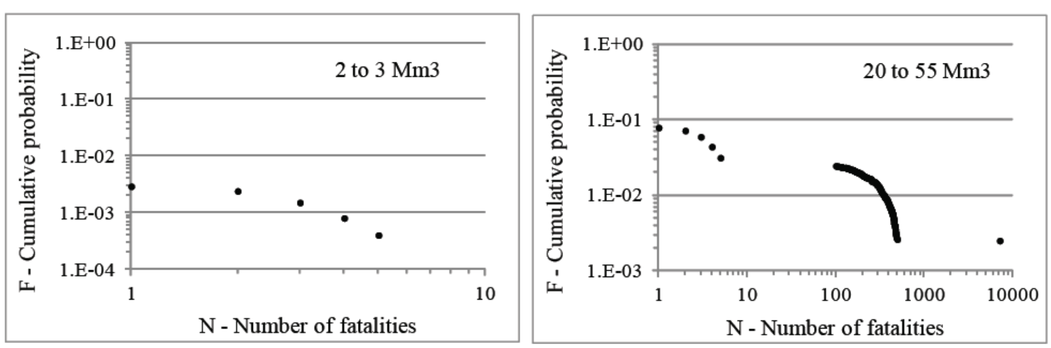


Figure 4-32 Cumulative conditional probability of fatalities given a slope failure scenario is realized (Left: 2 to 3 Mm^3, Right: 20 to 55 Mm^3) - Checkerboard Creek study.

4.3.3. Risk Estimation, Evaluation and Management

The consequences need to be combined with the slope failure probability to estimate the risk to life for the population at Revelstoke given a failure of the

Checkerboard Creek slope. Table 4-11 presents the failure probabilities for the two volume scenarios considered to potentially lead to loss of life in Revelstoke. These probability ranges are adopted following the results from the hazard analysis discussed previously in this chapter. A uniform distribution is defined between these range of failure probabilities and a Monte Carlo simulation technique is then applied. The results obtained are presented as cumulative density function F of N or more fatalities (Figure 4-33).

Volume	Subjective probability of occurrence	
	Maximum	Minimum
2 to 3 Mm^3	5E-2	1E-4
20 to 55 Mm^3	1E-5	1E-6

Table 4-11 Adopted slope failure probabilities after the hazard analysis - Checkerboard Creek study.

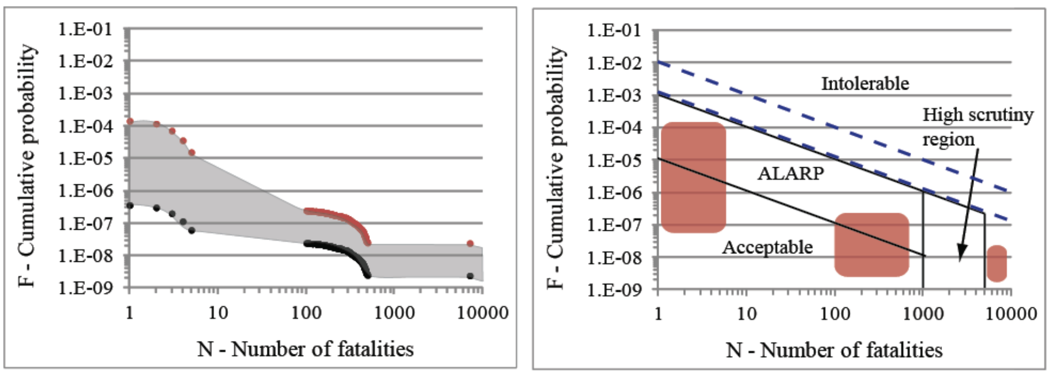


Figure 4-33 Estimated societal risk at Revelstoke associated with a failure of the Checkerboard Creek rock slope. Left: calculated risk maximum and minimum values. Right: Interpreted areas where the societal risk lies compared with the criterion adopted in Hong Kong (ERM 1998, in solid lines) and the criteria adopted by the U.S. Bureau of Reclamation (2003, in dashed lines) for expedited action needed to reduce risks (above upper line) and action needed to reduce risk (above lower line).

The calculated risk maximum and minimum values (left side on Figure 4-33) correspond to uncertainties in the slope failure probabilities and consequence analysis. These results were interpreted as areas representing the estimated societal risk and its uncertainty (right side on Figure 4-33). Also shown in the right side of this figure are the criteria adopted in Hong Kong for development in landslide prone areas (ERM 1998) and the criteria adopted by the U.S. Bureau of Reclamation (2003).

The societal risks lie below the tolerable threshold and partially above the acceptable threshold according to the criteria adopted in Hong Kong. This would suggest that measures should be taken to lower risks to the As Low As Reasonably Practicable levels (ALARP). Also, it would suggest that reduction of the uncertainties and/or conservatism in the analyses used for populating the risk estimation model is required to optimize the resources needed for risk mitigation.

The societal risk also lies well below the criteria adopted by the U.S. Bureau of Reclamation, which would suggest there is no need for further actions to reduce the risk levels. These criteria were chosen given the similar context of risk to life related to dam failures.

4.4. CONCLUSION

The risks associated with a failure of the Checkerboard Creek rock slope were estimated and evaluated against selected criteria. These criteria, however, would be applicable to risks associated with all foreseeable dam failure scenarios. Existence of other unstable slopes and the potential for a dam breach due to other mechanisms such as piping or flow overtopping would need to be summed up with the risks associated with the Checkerboard Creek in order to be compared against the criteria. Another approach is to apportion the risk criteria given the existence of other dangers.

It is interesting to note the vertical cut-off lines adopted by Hong Kong in their evaluation criterion. This corresponds to the society aversion towards events leading to a large number of fatalities. Given the population at Revelstoke, and the energy of a flooding event after a breach of the earthfill dam, there is a residual probability of an event leading to over 7,000 fatalities. Note also that the risk analysis did not consider risk mitigation strategies such as monitoring and early warning systems (which are currently in place). As such, the evaluated risk level corresponds to one where no specific risk mitigation is in place other than those related to the operation of the reservoir itself (freeboard heights).

It can be argued that increasing the resources to mitigate these risks can lower them to negligible levels, however there is the potential for a residual risk to always be present. This is one subject that QRA and risk management haven't been able to fully resolve. Risk mitigation against fast slope movements has been discussed by Morgenstern (2005). Some of the methodologies discussed are applicable, to some extent, to the Checkerboard Creek rock slope. These include avoiding rapid failure modes, and avoiding consequences by means of warning systems and by means of protective structures. Avoiding a rapid slope failure or building protective structures to prevent the failed slope from entering the reservoir - or against flooding of Revelstoke - such that the residual risk is lowered to nil, would require a prohibitive amount of resources. On an attempt to cope with these situations, particularly when dealing with large landslides, monitoring and early warning systems are typically adopted. The monitoring and early warning systems implemented at Turtle Mountain in the Province of Alberta, Canada (Froese *et al*. 2006) and the Åknes rock slope in Norway (Lacasse *et al*. 2008) are two examples of this approach.

The objective of early warning systems is to minimize the population exposed, thus avoiding large numbers of fatalities. However, warning needs to be timely, reliable, and communicated effectively (Morgenstern 2005). This requires a robust, reliable and redundant real-time monitoring system, as well as sufficient knowledge of the mechanisms and potential triggers leading to slope failure. It also requires an appropriate risk communication strategy. This communication strategy needs to ensure the timely initiation of emergency plans, transmit to the public the sense of risk posed by the hazardous slope, and explain the potential for false warnings and their cause.

Coupling between QRAs, early warning systems and the Observational Method seems to be one way forward to achieve a cost effective, robust and reliable risk management approach for large slopes. Figure 4-34 presents a simple chart showing one possible relationship for coupling these concepts. In this chart, QRAs are not only used to evaluate the current risk levels, but to measure its variability if changes in the slope behaviour and characteristics are noticed through monitoring. This requires the risk analysts to foresee these possible changes. It is acknowledged this increases the amount of effort required for analysis. However, even if done in a simplified manner and increasingly relying on expert opinion, a better understanding of the new threats can be gained and a more robust mitigation strategy achieved. Early warning and emergency plans are then linked to selected thresholds on the parameters being monitored. The chart, however, allows for these thresholds to be associated with changes in the estimated risk levels rather than a perceived increase in the probability of sudden failure.

The Observational Method is discussed in detail by Peck (1969). In essence, the method consists in developing engineering design in light of the available information. Calculations are done for the expected ground conditions. Modifications to the design are then proposed based on potential deviations from these assumed ground conditions. Instrumentation is then designed in order to detect these deviations by continuous monitoring of relevant parameters. In a risk management context, particularly for the chart presented in Figure 4-34, the Observational Method relates the adequacy of the risk mitigation strategy for the risk level estimated given the state of the slope, according to the monitoring information at the time. Foreseeing potential changes in the slope behaviour, noticeable through monitoring and related to a new risk level, would then lead to the application of previously assessed mitigation strategies that would lower the new risks to acceptable or tolerable levels. As an extreme, the decision for evacuation of a sector of the population could be included as one of these strategies, related to monitoring results indicating a large scale failure might be imminent.

Figure 4-35 shows a preliminary chart relating the potential slope deformation trend changes with changes in the risk levels and risk mitigation and management approaches. It is noted that if the observed changes are combination of the ones presented in this chart, the effects should be summed up. The nature of this chart is qualitative and a necessary first step for a comprehensive analysis to be achieved. The second step is to quantify the new risk level according to the implications that the slope behaviour change has in the input variables for risk estimation (an increase in deformation rate can increase our perception of the failure probability, or reactivation of previously stable areas would increase the volume of the slope failure scenarios analysed). This changes can be linked to changes in the failure mechanisms, kinematics, potential triggers or indications that the slope is reaching imminent failure.

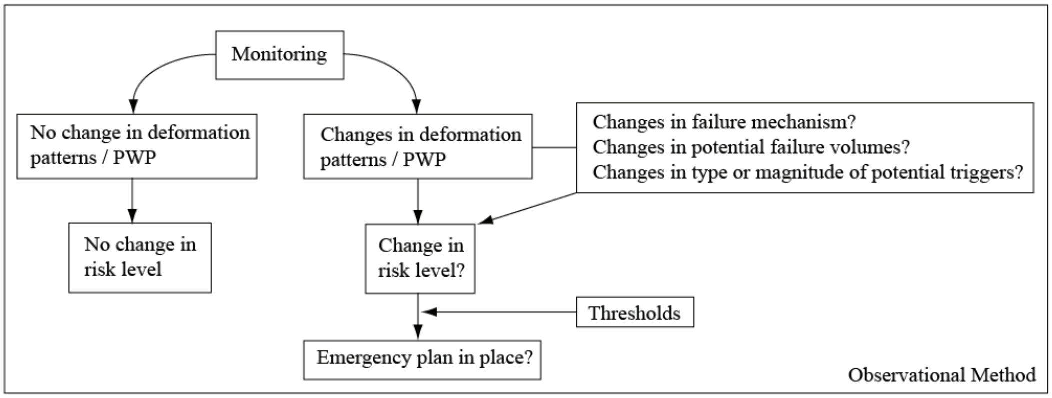


Figure 4-34 Coupling between QRA, early warning systems and the Observational Method for risk management of large slopes.

The new risk levels then need to be evaluated against the selected criterion. This new evaluation and the insight into the likely causes for the slope change in behaviour, aid in defining cost-effective risk mitigation strategies for the foreseen slope conditions.

Finally, threshold values need to be defined for the variables being monitored (such as ground water levels and deformation rates) and for changes in the deformation patterns such as extent of reactivated areas. These thresholds will define when the observed slope change in behaviour should be linked to a change in risk level, and a risk mitigation/management strategy adopted. An example of this is the selection of deformation thresholds linked to increasing warning levels and emergency plans. These thresholds should be developed for each particular case and periodically revised in light of new information.

It is noted here that observation should not only apply to changes in the slope behaviour and conditions, but also changes in the characteristics of the elements at risk and the surrounding environment (population density, reservoir levels, new infrastructure, stabilization works, weather events and climate change). The discussion presented here implies an increase in the amount of analysis that may seem overwhelming. It is believed, however, that as QRAs become more comprehensive and systematic, this is a potential way forward for strengthening the methodology. For illustration purposes, the changes in risk level and mitigation strategy are presented and discussed for two scenarios of change in Checkerboard Creek rock slope deformation pattern.

Change in the slope deformation pattern	Description	Change in our understanding of the slope deformation mechanisms and potential triggers	Change in the failure scenario volumes and perceived likelihood of sudden failure	Change in the likelihood of dam overtopping and dam breach	Effects on the risk to life at Revelstoke	Risk management aproach
None	Extent of active deformation zone remains the same. Deformation rate also remains the same. Deformations only occur during the cooling periods between October and April.	None	None	None	None	Continue the observation of the slope behaviour.
Rate increase	Deformation rate increases.	The deforming mass could be transitioning from a state close to limit equilibrium towards an unstable state and subsequent sudden failure.	The slope is likely to be closer to a state of limit equilibrium. The preceived likelihood of sudden failure increases.	Increase only as a consequence of increased likelihood of a slope sudden failure.	Increase	Assess the need for evacuation based on established criteria.
Stable area re-activated	Slope deformations are measured in areas or at depths where no previous deformations were measured.	Actively deforming mass has increased or is increasing. Possible reasons could be progressive degradation of the rock mass due to continuous deformation and related loss of confinement.	The volume of a potential sudden failure increases.	Increased as a consequence of an increase of the potential slope failure volumes.	Increase	Continue the observation of the slope behaviour. Deformation mechanisms and magnitude of consequences need to be revised. Update the risk assessment and revise established criteria for evacuation in light of the new information. Assess other potential risk mitigation strategies.
Ongoing deformation throughout the year	Deformation is measured during the current active period (October through April) and continues through all or part of the current quiet period (May through September).	At the time, deformations are related to loss of confinement due to contraction related to temperature decrease. Deformations measured during the warming period could correspond to a change in the deformation mechanism, where the deforming mass is closer to a limit state of equilibrium. Potential triggers such as snow pack and transient water pressures could become more likely to cause a sudden failure of the slope.	The preceived likelihood of failure increases.	Increase only as a consequence of increased likelihood of a slope sudden failure.	Increase	
Formation of basal slip surface	Deformation pattern starts showing a concentration of displacements along a basal shear zone.	Ongoing deformation of the highly desegregated mass could eventualy lead to the formation of a basal shear zone that would allow the deforming mass to slide through it. Transient water pressures within this shear zone could then become potential triggers of sudden failure.	The most likely failure scenario could be defined by the developed shear zone. Limit equilibrium models and return periods of potential triggering mechanisms can aid in estimating the probability of sudden failure.	Will be related to the new deformation pattern and volumes defined by the shear zone. Post-failure velocities are also likely to change, leading to diferent wave heights for the new scenario.	To be defined based on new deformation mechanism.	
Elevated PWP within the deforming mass	Pore water pressures start to increase within the deforming mass in a regular basis or remain elevated as oposed to the current observations where the deforming mass is virtually drained.	Elevated pore water pressures could mean the deformation of the slope mass has eventually blocked major drainage paths within the slope. Seasons with precipitation events above normal could become potential triggers as well as larger amounts of snow acumulations at the time of spring meltdown.	The preceived likelihood of failure increases.	Increase only as a consequence of increased likelihood of a slope sudden failure.	Increase	

* If the slope behaviour change is a combination of any the above, the implications of each type of change observed should be considered.

Figure 4-35 Potential slope deformation trend changes related to risk levels and risk mitigation and management - Checkerboard Creek study.

Scenario 1: 2 to 3 Mm^3 mass increases its deformation rate to that of the currently fastest moving sections (10 to 15 mm/year) and shows deformation throughout the year.

Given monitoring shows the actively deforming mass to reach a deformation rate similar to those measured at the fastest deforming sections (10 to 15 mm/year), it is argued that the perceived likelihood of failure of such volume will increase significantly. This will in turn increase the estimated probabilities for certain wave overtopping heights leading to increases in the life loss probabilities after flooding at Revelstoke. It is judged the failure probability of a mass 2 to 3 Mm^3 in volume to be increased to about 10^{-1} to 10^{-2}. Figure 4-36 presents the estimated societal risk under this scenario and compares it to the selected criteria.

The societal risks under this scenario would be considered not to required action for risk reduction according to the U.S. Bureau of Reclamation. They lie within the ALARP region according to the criterion adopted in Hong Kong. It could then be decided these risks should continue to be tolerated. The mitigation approach would not change in such case (observation of the slope behaviour linked to early warnings to minimize the exposed population would remain in place - Figure 4-35). Not being able to link this slope behaviour change to the absolute estimated risks could lead to unnecessary evacuation of a large population.

If the risks are thought to need reduction, mitigation strategies can include the increase in reservoir freeboard (thus decreasing the overtopping heights), buttressing of the slope and realignment of the highway, and flood control structures at Revelstoke. These should be decided upon selection of a method to assess if the ALARP principle is being met, among other technical, social and economic factors. In any case, the new levels of risk achieved after these mitigation strategies are in place need to be estimated and evaluated. Given the costs associated with these mitigation options, continued monitoring and early warning systems is typically the option of choice to reduce societal risks.

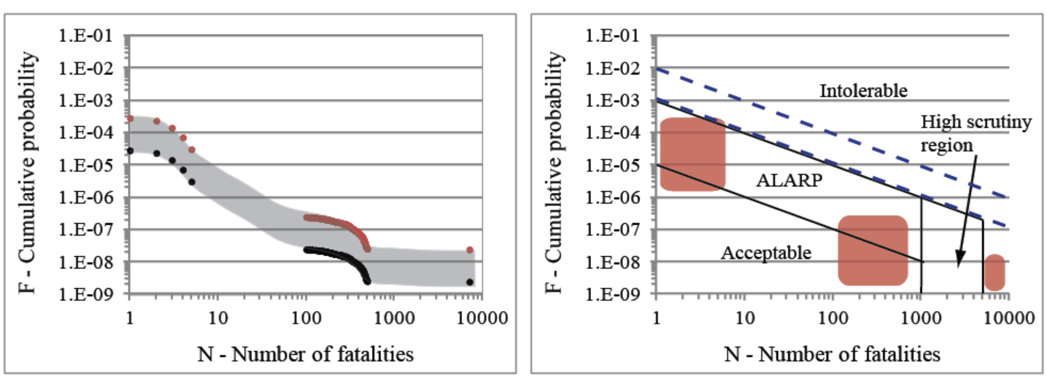


Figure 4-36 Left: Estimated societal risks posed by the Checkerboard Creek rock slope for the case where 2 to 3 Mm^3 mass increases its deformation rate to that of the currently fastest moving sections (10 to 15 mm/year) and shows deformation throughout the year. Right: Interpreted areas where the societal risk lies compared with the criterion adopted in Hong Kong (ERM 1998, in solid lines) and the criterion adopted by the U.S. Bureau of Reclamation (2003, in dashed lines) for expedited action needed to reduce risks (above upper line) and action needed to reduce risk (above lower line).

Scenario 2: Entire Checkerboard Creek rock slope (20 to 55 Mm^3) starts showing shearing along a basal slip surface.

This is considered an extremely unlikely scenario, however it was chosen for illustrative purposes. Given monitoring shows that the entire Checkerboard Creek rock slope (20 to 55 Mm^3) starts showing shearing along a basal slip surface, the perceived likelihood of failure of such volume will increase. Increases in the probabilities for certain wave overtopping heights will lead to an increase in the earthfill dam breach probability, and life loss probabilities after flooding at Revelstoke will also show a significant increase. Given this scenario, it is judged the failure probability of a mass 20 to 55 Mm^3 in volume to be in the order of 10^{-2} to 10^{-4}. The slide velocity PDF could also change. Figure 4-37 presents the estimated societal risk under this scenario and compares it to the selected criteria.

In this case the societal risks start moving into the intolerable regions for both criteria. Options for active risk mitigation would be similar to those described for the previous scenario. The magnitude of the slope deforming mass would make the costs associated with these options prohibitive, unless drainage becomes an option given the change in slope failure mechanism and the reservoir levels. Here, continued monitoring and early warning systems would also likely be the option of choice to reduce societal risks. However, given the high risks shown in the assessment, quantification of risk reduction due to monitoring and early warning becomes unavoidable.

Quantifying risk reduction after monitoring and early warning systems are in place is not an easy task. It has long been recognized that the onset of sudden slope failures are preceded by changes in the deformation pattern, particularly an increases in the deformation rates (Ter-Stepanian 1963, Leroueil *et al*. 1996, Rose and Hungr 2006). This characteristic is now used to predict imminent slope failure and a basis for warning systems (Zavodni and Broadbent 1978, Fukuzono 1985, Zavodni 2000, Crosta and Agliardi 2003, Morgenstern 2005, Rose and Hungr 2007). In controlled environments such as the open pit mining industry, where the number of exposed workers and equipment is limited and workers are required to follow safety instructions, early warning systems can effectively lead to timely evacuation and consequence reduction. When the population at risk is the general public, and for a large exposed population, factors such as an effective risk communication, the warning being issued efficiently, evacuation procedures being carried out as planned and the people response towards an imminent catastrophe (just to name a few) will play a major role in the effectiveness of the early warning system.

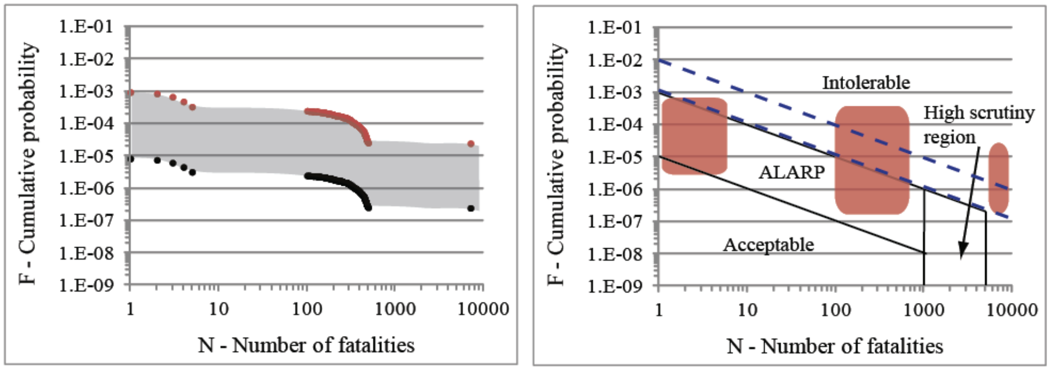


Figure 4-37 Left: Estimated societal risks posed by the Checkerboard Creek rock slope for the case where the entire Checkerboard Creek rock slope (20 to 55 Mm^3) starts showing shearing along a basal slip surface. Right: Interpreted areas where the societal risk lies compared with the criterion adopted in Hong Kong (ERM 1998, in solid lines) and the criterion adopted by the U.S. Bureau of Reclamation (2003, in dashed lines) for expedited action needed to reduce risks (above upper line) and action needed to reduce risk (above lower line).

Some approaches have been developed to estimate life losses related to flooding events considering the effects of warning times and evacuation success levels (Brown and Graham 1988 for the U.S. Bureau of Reclamation on a semi-quantitative manner, Peng and Zhang 2012 developed a quantitative procedure using Bayesian networks). These, however, are associated with much uncertainty related to environmental conditions, public response, distance from the hazard and to a safe zone, resources available for evacuation, and knowledge of the evacuation routes and procedures. The success rate of early warnings needs to be assessed for each specific case, and ideally calibrated by measuring the time and number of people evacuated during evacuation drills.

Regarding the adoption of early warning systems for risk mitigation, there is the risk associated with false warnings undermining the public trust in the system and dramatically lowering the evacuation success rate. Adequate and transparent risk communication, explaining the uncertainties inherent to the phenomenon, the risk levels and the potential for false indications of sudden failure; should aid in reducing this risk.

CHAPTER 5: DEVELOPMENT OF RISK TO LIFE EVALUATION CRITERIA

5.1. INTRODUCTION

The pressure on the geotechnical engineer to apply quantitative risk assessments has kept increasing since Morgenstern highlighted this trend in 1997. This is particularly the case of regulatory agencies including the use of quantitative methods within published guidelines, and clients willing to improve the efficiency of their risk management procedures. One example of this is the initiative by the Australian Geomechanics Society to develop risk management guidelines emphasizing quantitative methodologies. This was in response to the increasing request of local authorities for risk assessments to be developed prior to urban development of new areas (AGS 2007). The province of British Columbia in its Guidelines for Legislated Landslide Assessments for Proposed Residential Developments in BC (APEGBC 2010) also considers the application of quantitative assessments for land use planning. Another example are the efforts by the railway industry in North America to develop comprehensive risk management strategies, which are consistently leading towards more detailed quantitative analyses of sections considered critical given their high hazard levels.

As noted by Morgenstern in his Casagrande Lecture (1995), the full potential of quantitative risk analyses is met when evaluated against adopted criterion. Moreover, quantitative risk analysis alone has limited benefits (Leroi *et al*. 2005) and evaluation is deemed as an important stage of the framework (Fell 1994). The advantages of quantitative evaluations mainly lie in 1) the possibility of assessing the risks levels in absolute terms, 2) the comparison and integration of mitigation strategies through quantitative estimation of risk reduction is highly improved when compared against adopted criterion, thus assessing the most cost effective strategy for which the goals are met, and 3) provides a framework for more objective and transparent decision making that can be shared with regulators, stakeholders and the population.

However, when risk levels are to be measured in terms of the probability of lives being lost, development of evaluation criteria is by no means an easy task. Establishing acceptable limits to life loss probability is not a scientific matter alone, and decisions on the evaluation criterion involve considerations of legal, political, social and financial issues (Fell 1994, Ho *et al*. 2000). In this regard, final decision on the criterion to be adopted corresponds to the regulator or client. Being informed about the precedents, details and limitations of the analyses, risk analysts need to be active parts in the development of proposed risk evaluation criteria (Leroi *et al*. 2005). They should provide the decision-makers with the necessary information in which to base any tolerable or acceptable risk threshold adopted.

Given the difficulties associated with the adoption of risk evaluation criteria, it can be appealing to adopt previously proposed ones. However, the contexts for which these criteria were developed differ, and their applicability to any specific situation should be assessed before adopting them. In this regard, this chapter summarises the main considerations for developing risk evaluation criteria and proposes a framework for defining these criteria. Two simplified examples on the development of proposed risk evaluation thresholds are also presented. It is noted that the framework can be used to assess the applicability of previously proposed criteria to a specific context.

5.2. TERMINOLOGY

The terminology used in this chapter follows that presented by The International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) Technical Committee on Risk Assessment and Management (TC32) in their Glossary of Terms for Risk Assessment, and reproduced in Fell *et al*. (2005). Some of the terms referred to in this chapter were defined in Chapter 2. Two terms are introduced in this section in order to highlight the difference between them and their definitions are quoted after Fell *et al*. (2005):

> "Acceptable risk: A risk which everyone impacted is prepared to accept. Action to further reduce such risk is usually not required unless reasonably practicable measures are available at low cost in terms of money, time and effort.
>
> Tolerable risk: A risk within a range that society can live with so as to secure certain net benefits. It is a range of risk regarded as non-negligible, and needing to be kept under review and reduced further if possible."

The state of the art framework for landslide risk management is summarized in Chapter 2. Also, for purposes of the present discussion, a system is defined as the hazard (the slope with the potential to fail) and its interaction with the infrastructure and population exposed to it (transportation corridor, dam operation, urban development).

5.3. PREVIOUSLY PROPOSED CRITERIA

There are a handful of criteria for risk evaluation that have been proposed outside and within the geotechnical community, and that have become common practice in geotechnical related risk assessments. The most widely used are the criteria developed by the Health and Safety Executive (HSE) in the United Kingdom for land use planning around industries (HSE 2001), the Australian National Committee on Large Dams (ANCOLD) for population exposed to potential dam failures (ANCOLD 2003), the United States Bureau of Reclamation, also for people exposed to potential dam failures (U.S. Bureau of Reclamation 2003); and the Hong Kong Special Administrative Region Government for land development in landslide prone areas (ERM 1998). In the last decade, the Australian Geomechanics Society has developed and updated their suggested guidelines for landslide risk management, which sets a common framework within the

Australian states and territories for risk analysis and evaluation of landslides (AGS 2007). These criteria have been discussed in Finlay and Fell (1997), ERM (1998), Leroi *et al*. (2005), Ale (2005), Porter *et al*. (2009) and Scarlett *et al*. (2011), among others, and it is not the intention to discuss their details here. The thresholds adopted by these criteria are presented later in the chapter.

The differences in context for which these criteria were developed arise from the nature of the hazards, the extent of the system to be regulated, the characteristics of the elements at risk and their social, economic, political and cultural environment. The HSE proposed criteria to regulate land use planning around industries in the UK (HSE 2001). Their early publications indicate a focus on the risks posed by nuclear power plants, where low probability accidents are associated with high consequences which include not only immediate and short-term life loss, but also the possibility of significantly shorter life expectancy. In this context, consequences are thought of potentially involving regional scales, long lasting, and newly imposed (HSE 1992). In contrast, slope failures in developments within landslide prone areas, which is the context of the criterion developed in Hong Kong (ERM 1998), are seen as characteristic of the region, localized, and with no long term consequences involving loss of life. There is a sense of higher tolerance towards these risks (Fell 1994, Finlay and Fell 1997).

Another important aspect is the fact that Hong Kong is densely populated and characterized by its hilly terrain. As such, development of areas outside the influence of potential landslides is not always and option. Also, the high demand for space leads to high density housing to be opted. In such a context, even when there is a long history of landsliding (ERM 1998), too conservative approaches towards development regulations would lead to significant economic losses related to limitations in land use. This should be taken in consideration if the criteria are to be adopted in regions where population density is low and areas for new development can be found easier.

A different context is associated with the criteria developed by both ANCOLD and the U.S. Bureau of Reclamation. These aim to regulate the risks associated with the presence and operation of dams, a man-made structure, which includes the dam itself, the reservoir, and all potentially unstable slopes within. Risks posed by potential dam breaches under different scenarios need to be integrated before these can be assessed against the proposed criteria. In particular, these criteria should not be used in isolated risk assessments of potential slope failures within dam reservoirs. It could be the case that risks related to each potentially unstable slope lie below the threshold values for the adopted criterion, however, risks lie above the tolerable limits when integrated. When assessing the risks related to a particular slope (which is a common practice when the slope is considered critical), the criterion adopted should reflect the fact that other hazards can lead to a dam failure (other unstable slopes, failure of the dam structure, extreme weather events).

The variability in the context where these criteria were proposed suggests that the development of risk evaluation criteria is best suited when defined at regional, industry, client and even case specific scales. These criteria can be helpful as starting point for the development of risk evaluation thresholds, given the context of the system matter of analyses is similar. In any case, the applicability of previously proposed criteria should be assessed before it is adopted.

5.4. A FRAMEWORK TO DEVELOP RISK EVALUATION CRITERIA

5.4.1. Initial Considerations

Adoption of risk acceptability thresholds will be influenced by the social, political, economic and cultural context (Rowe 1977, Fell 1994, Fell *et al*. 2005). Detailed discussion of some of these issues is presented in Rowe (1977) and Leroi *et al*. (2005). It is clear that these issues are far beyond the responsibilities and expertise of the risk analyst. Following the position in IUGS (1997), it is not the intention to establish any particular criterion. This should remain responsibility of owners, regulators and governments. However, it is deemed essential for the risk analyst to be involved in developing the criterion, and for their input to be clear and follow an auditable framework.

As shown in Figure 5-1, any framework for the development of risk to life evaluation criteria needs to consider three main aspects. The first aspect, the characteristics of the system being analysed, is technical in nature. It considers the hazard characteristics, such as extent and type, and its interaction with the exposed population (development in landslide prone areas, landsliding within reservoirs, ground hazards along transportation corridors). The second aspect is the social, political, economic and cultural context. This aspect is fundamentally of social nature and the proposed framework leaves these to be considered by the owner or regulator.

The third aspect, the principles for developing risk evaluation criteria, has a philosophical nature. In summary, it mainly consists of the attitude towards risk tolerance considering the exposed population class (workers, users, public), if it is an existing hazard or will be newly imposed, and if the associated risks are thought as voluntary or involuntary. Diverse fundamental principles to develop risk criteria have been discussed in the literature (Pandey and Nathwani 2004, Skjong *et al*. 2005, Vanem 2012), and their details will not be replicated here.

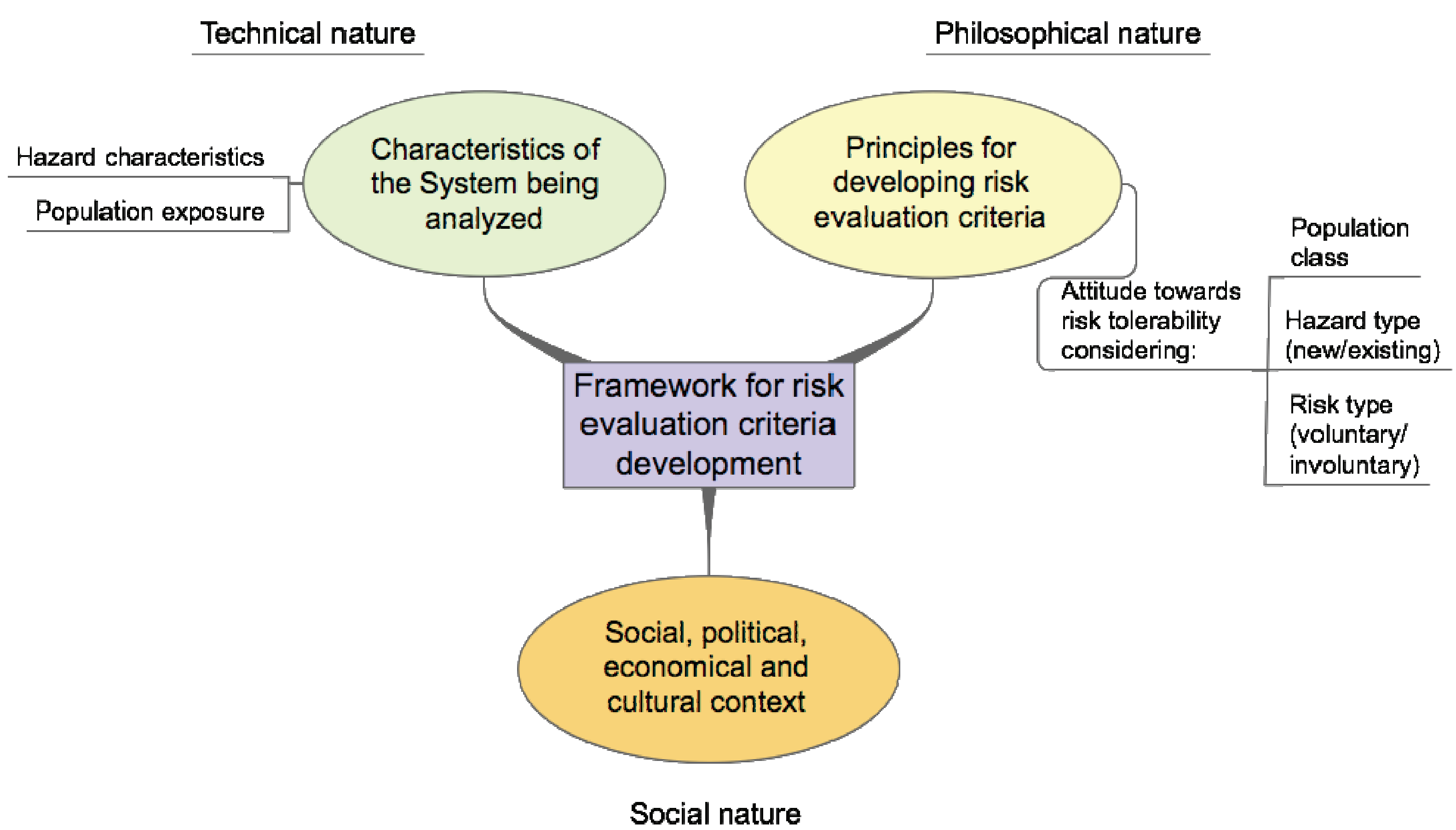


Figure 5-1 Aspects to be considered when developing proposed risk to life evaluation criteria.

Selecting the principles that would lead to a clear framework for risk criteria development should address three important considerations: 1) there is a need to regulate risks posed by natural and cut slopes in a sound, clear and consistent manner, 2) risk thresholds aid in evaluating the real urgency for mitigation strategies, risks deemed as not tolerable would require mitigation to assure a minimum quality of life, 3) uncertainties inherent to a system can be dealt with by assessment and management of its associated risks, thus maximizing the benefit for the parties involved. Then, the following combination of principles is selected. A brief description is also presented for each, after Vanem (2012):

Absolute risk criterion: The level of risk itself is studied and the risk criterion is formulated as a maximum level of risk that should not be exceeded, without regard to the cost and benefit associated with it.

The ALARP principle: Risks should be managed to be As Low As Reasonably Practicable (ALARP). Both risk levels and the cost associated with mitigating the risk are considered, and all risk reduction measures should be implemented as long as the cost of implementing them is reasonably practicable according to cost effectiveness considerations.

The principle of equivalency: Risk should be compared with known levels of risks from similar activities or systems that are widely regarded as acceptable or tolerable, to require that an equal level of risk be obtained. Similarly, comparisons can be against historic data, natural disasters, and life expectancy.

The accountability principle: Transparent and clearly defined criteria, which should be quantitative rather than qualitative and based on objective assessments (as far as possible) rather than subjective interpretation of risk. The formulation of the criteria should be explicit, rendering little room for different interpretations of the evaluation criteria themselves.

The holistic principle: Decisions regarding safety on behalf of the public should be based on a holistic consideration of all risks and apply across the complete range of hazards. Only when the total risk the public is exposed to is properly assessed, can the proposed risk reduction measures be evaluated and risk criteria established. Given the difficulties and effort this would require, the principle is applied at the scale of the system being analyzed and requires simplification (such as apportioning and scaling).

Principle of parsimony: Simpler risk acceptance criteria might be preferable to complex ones. It is important that the criteria and procedures are simple enough to be practical and facilitate communication.

The framework should be consistent with these principles and with common practice in other industries. As such, it considers proposing threshold values for acceptable and tolerable risks. Risks above the tolerable threshold are considered not tolerable and risk mitigation is mandatory. An ALARP region is placed below the tolerable threshold. If the owner / regulator decides on adopting an acceptability threshold (risks below this threshold need no further reduction), the

ALARP region lower boundary is determined by it. Else wise, the ALARP concept is applied to all risks lying below the tolerable threshold.

The framework also considers risks to be assessed in terms of the individual risk (for the individual estimated to be at highest risk) and societal risk (through F-N plots). F-N plots consist of log-log plots with the number of fatalities on the horizontal axis (N) and the probability of N or more fatalities on the vertical axis (F). The proposed thresholds for risk acceptability and tolerance are drawn as lines with negative slopes to show risk aversion towards accidents involving large number of fatalities. F-N plots are common practice in industries such as nuclear-power generation, land use planning around industries, dam operations, maritime industries, and land development in landslide prone areas (HSE 1992, Morgenstern 1995, Ho *et al*. 2000, HSE 2001, Leroi *et al*. 2005, Skjong *et al*. 2005, Porter *et al*. 2009, Scarlett *et al*. 2011). A point in the F-N plot (anchor point) and a slope of a line are required to building a risk threshold line. The anchor point is a threshold cumulative probability for a certain number of fatalities, typically N=1. The slope of the line is typically defined between -1 and -2, being -1 mostly adopted. Skjong (2002), Ale (2005), Skjong *et al*. (2005), CCPS (2009), and Vanem (2012) discuss the implications of the slope chosen. Adoption of this methodology is in line with the principle of parsimony, as it has been shown to aid in the visual communication of risks and to be of common use.

5.4.2. Proposed Framework

The proposed framework is presented in Figure 5-2. It has been divided in three major stages according to the major participants that should lead the analyses. Each stage then consists on a few steps, which should be followed in sequence (see numbering in Figure 5-2). The first stage is considered to be carried out through a coordinated effort between the risk analyst and the owner / regulator, and needs to be consistent with the risk analysis scope of work. The three steps proposed should be thought of while defining the output of the analysis (or how is risk to be measured), as the criterion and estimated risks need to be compatible.

The population within any system can be grouped in three classes 1) Workers, which are people exposed to the hazard in exchange of economic or professional gain, 2) Users, which are people exposed to the hazard in exchange of a gain or to cover a need, and 3) Public, which are people exposed to the hazard not being aware of any direct benefit in exchange. This grouping is fundamentally about defining voluntary and involuntary risk levels. This grouping will then be used for characterizing the type of risk (next stage). As previously discussed, the framework proposes to assess the risks in terms of individual and societal risks. However, there can be systems where societal criteria are not necessary, such as systems where the maximum number of people exposed is small (railway freight crew members usually travel in groups of 2 or 3).

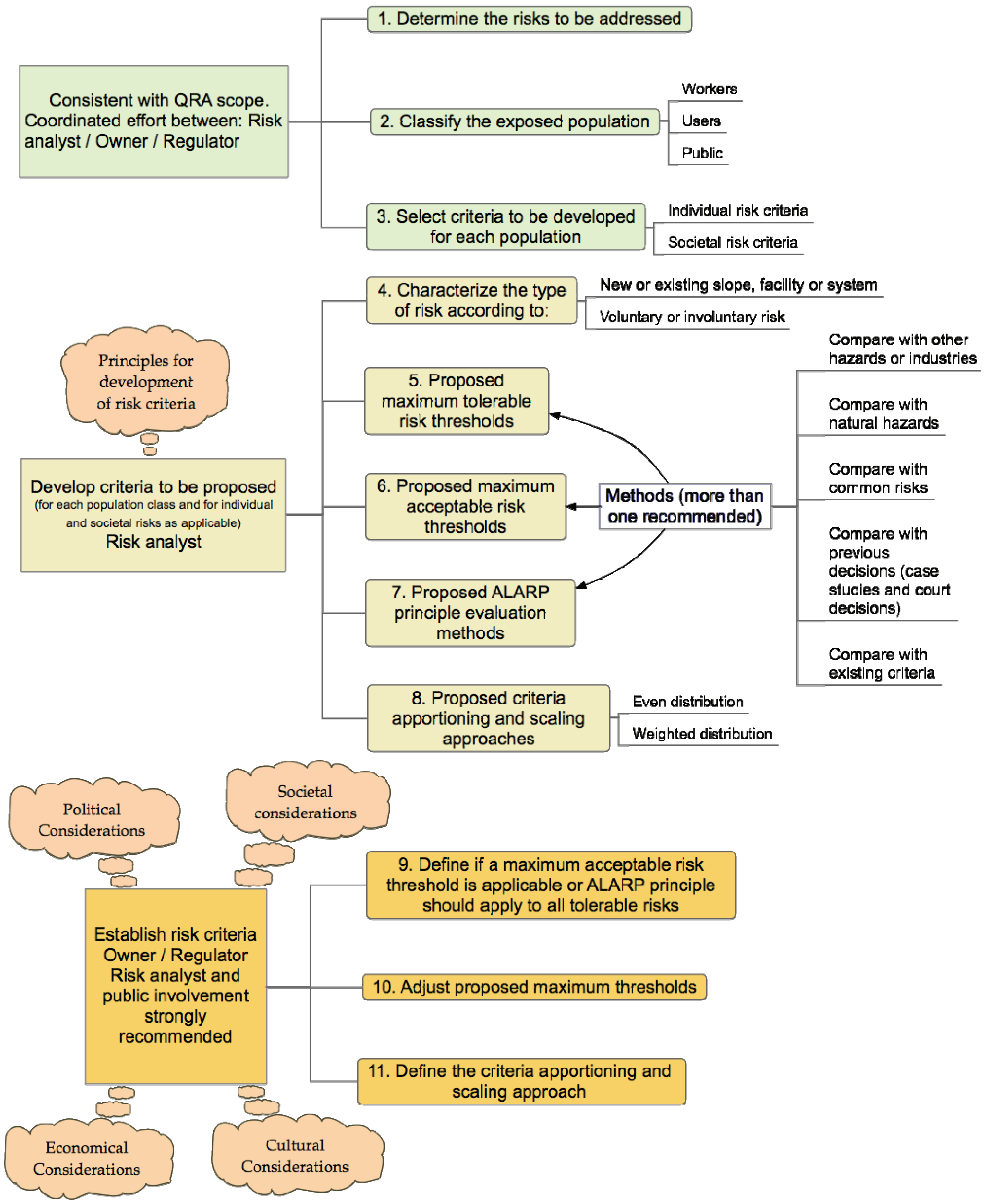


Figure 5-2 Proposed framework for the development of risk to life evaluation criteria.

It is proposed the second stage to be carried out by the risk analyst given their acquired knowledge of the entire system. The steps in this stage need to be followed for each population type and for individual and societal risks, as applicable. The steps, assumptions, and principles behind the development of the proposed criterion need to be clearly stated, as they will be reviewed in the later stage by the owner / regulator and, ideally, the population exposed. When possible, more than one option for the proposed thresholds should be presented. In that case, the differences in the fundamental thinking behind each proposed threshold should also be stated. The second stage should start with a characterization of the risks according to 1) is it a new or existing slope, facility or system. This includes differentiating between man-made structures (road cuts and reservoirs) and natural slopes, and 2) is the risk voluntary or involuntary, which follows the population classification in the first stage. The remainder steps (5 through 8 in Figure 5-2) consist on developing the proposed risk thresholds, the ALARP principle evaluation criterion, and how to apportion and scale the proposed criterion within the system, when applicable. Methods to achieve these are presented later in this chapter.

The last stage is the responsibility of the owner / regulator. It consists of making final decisions and adjustments to the developed criterion considering the social, economic, political and cultural context. The risk analyst acts only as a consultant and it is strongly recommended for the public to be involved in the decision-making. This implies simple and clear explanations of the risk analysis process, its limitations, and the development of the criterion as well as the principles behind it. It will require a good risk communication strategy and proper risk/benefit distribution among the population exposed. These later issues are beyond the scope of the present discussion.

The risk criterion should be reviewed and periodically updated in light of changes in both the hazard (stabilization, changes in technology, new measurements) and the elements at risk (changes in public expectations, exposure).

5.4.3. Methods for Developing Proposed Risk Thresholds

There are several methods for developing proposed risk thresholds, which are subject to continuous debate in the literature. Methods available are (after Morgenstern 1995, Skjong 2002 and CCPS 2009):

5.4.3.1. Comparison with Statistics Within the Industry or for Similar Industries and Activities

This method implies that, if the industry or activity chosen for comparison is currently taking place, the risks associated with it are considered in balance with the benefits gained. The activity chosen needs to reflect the same risk characteristics as the system being assessed (voluntary or involuntary). For example; the railway passenger industry should be as safe as the air passenger transportation industry.

5.4.3.2. Comparison with Natural Hazards

This method consists of comparing the system to statistics on lives lost due to natural events. It implies that the risks we impose on ourselves should be a small portion of what can be blamed on nature. For example; the annual risk to the public imposed by the slope cut should be equal or less than the risk associated with thunderstorms.

5.4.3.3. Comparison with Common Risks

This implies the imposed risks by the system analyzed should not be greater than risks from common activities such as swimming and driving. Example: The risks associated with landsliding along a section of the highway should be less than those associated with crossing the street or driving the highway.

5.4.3.4. Comparison with Previous Decisions

In this method, owners, regulators or court decisions on cases where the risks involved can be estimated are taken as indicative of society's tolerance of a particular activity or industry.

5.4.3.5. Comparison with Existing Criteria

In this method, previously proposed criteria for systems reflecting risk characteristics similar to the ones being assessed are used for comparison or validation. Example: Tolerable risks from landsliding for land development areas should be similar to those proposed in Hong Kong.

When developing proposed risk thresholds it is important to keep in mind the considerations presented by IUGS (1997). These considerations are consistent with the principles presented here and can be summarized as follows:

- Incremental risk associated with the system analyzed should not be significant when compared to the risks associated with everyday life, and whenever possible the ALARP principle be applied,
- Events with the potential to cause a large number of fatalities should have low occurrence probabilities. This accounts for society's lower risk tolerance of large numbers of fatalities,
- Some populations tolerate higher risks than others in relation to the required efforts to mitigate those risks, as they benefit from their existence or the activity realized (workers in the mining industry as opposed to the public), and;
- Tolerable risks are higher for natural slopes than for those engineered or controlled (slope cuts, earth fill dams, or even for natural slopes that are known to be monitored).

5.4.4. Methods for Evaluation of the ALARP Principle

One method to evaluate if the ALARP principle is being met is through conventional cost-benefit analyses. Here, the costs of implementing risk reduction measures are compared against the reduced risks in monetary units, thus assigning an economic value to life. Some estimates of the economic value of life appear in the literature, mainly considering people as a resource in an economic activity, however this approach conflicts with ethical traditions (Skjong 2002).

A preferred method is the use of a cost-effectiveness analysis (Skjong 2002, Skjong *et al*. 2005). This method calculates a ratio of the cost of implementing risk reduction measures to the reduction in risk, thus avoiding putting an economic value to life. According to Leroi *et al*. (2005) two commonly used variants of the method are the adjusted cost-to-save-a-statistical-life (ACSSL) and the unadjusted cost-to-save-a-statistical-life (UCSSL). These are analogous to the gross-cost-of-averting-a-fatality (GCAF) and the net-cost-of-averting-a-fatality (NCAF) respectively, as presented in Skjong (2002) and Skjong *et al*. (2005). These can be defined as (Leroi *et al*. 2005 p. 167):

$$\mathrm{ACSSL} = \frac{C_A - (E[\mathrm{bef}] - E[\mathrm{aft}]) - (O[\mathrm{bef}] - O[\mathrm{aft}])}{L[\mathrm{bef}] - L[\mathrm{aft}]}$$

$$\mathrm{UCSSL} = \frac{C_A}{L[\mathrm{bef}] - L[\mathrm{aft}]}$$

Where:

C_A is the annualized cost of implementing the risk reduction measure in dollars per year, $E[\mathrm{bef/aft}]$ are the economic risks (failure probability times monetary loss) in dollars per year before and after implementing the risk reduction measures, $O[\mathrm{bef/aft}]$ are the annual operational costs before and after implementing the risk reduction measures, and; $L[\mathrm{bef/aft}]$ are the estimated risk to life in lives per year before and after implementing the risk reduction measures.

When implementing this method for evaluation of individual risk, the value of L is the value of risk (annual probability of the individual being killed). When evaluating societal risk, the value of L can be estimated as the total risk, $\sum f_i . N_i$ where f_i and N_i are the annual probability (or frequency) and the correspondent number of lives lost, respectively.

This method is also useful for comparison of different risk mitigation options. More expensive options give larger CSSL (cost-to-save-a-statistical-life, either adjusted or unadjusted) values, while more effective options will give lower CSSL values (larger reductions in risk). However, assessing if the ALARP principle has been satisfied requires a criterion regarding the CSSL values considered cost-effective. Deciding what is considered to be cost effective CSSL values is not a simple matter. Risk reduction will have a direct economic impact on the owner, which could ultimately lead to an activity not being profitable. As a consequence, the workers might also be economically impacted (loss of jobs or income reduction). Where the regulator is also responsible for safety, thus having to pay for the risk reduction measures, the public is economically affected (their taxes pay for the reduction of risk).

Given these complexities, it is suggested that for estimated risks falling within the ALARP region, the risk analyst estimate the CSSL values for the risk mitigation measures proposed as part of the risk management process and final decisions should be left for the owners and regulators. When presenting the information, not only the CSSL ratios should be included, but also the actual increment in cost associated with the mitigation strategy and the estimated risks after their implementation. These absolute values are as useful as the CSSL for the decision making process. The participation of the public through surveys or public meetings should be encouraged, as they are part of the exposed population and are likely to be impacted economically by the decisions.

5.4.5. Methods for Apportioning and Scaling Criteria

Risk criteria are often defined for a certain scale of the system. These criteria need to be adjusted to reflect the scale of the particular system evaluated, or have to be apportioned throughout the sub-systems for their individual evaluation (for example risks at a specific mileage as opposed to the entire transportation corridor). If the criteria are not scaled or properly apportioned, higher risks than desired could end up being tolerated. A detailed discussion is presented in CCPS (2009).

An example is used to illustrate the concept. Typical systems analyzed are new developments within landslide prone areas or existing/proposed alignments of highways. Risk criteria can then be defined for the system (either the new development or the highway). The risk value to be evaluated against the selected criterion is the integration of the risks associated with all hazards and sectors within the system (all slopes, all potential failure volumes, every mile along the highway, and considerations of all other ground hazards). In practice, however, it is unlikely that all hazards and sectors be considered due to time constrains, budget limitations, or scenarios deemed negligible. To account for the different sectors (mileage along the highway), the criterion needs to be apportioned. If the apportioning is based only on the linear or aerial extent of the system, (same risk thresholds for each mile along the highway or for each slope within the proposed development), it is an even apportioning or distribution. It is often the case, however, that a number of sectors will be more hazardous than others. If the apportioning is even, the criterion could end up being too strict at some locations and not a reflection of the defined overall criterion. Apportioning the criterion with considerations of the relative hazard levels or the exposure of the population would be a weighted apportioning or distribution.

Both the individual and societal risk criteria should be apportioned. However how to apportion them can differ and be based on different considerations. Apportioning criteria should depend on the relative hazard levels and the exposure of the population (residential areas within the proposed development show dense populations spending up to half their time in the location in contrast to recreational areas such as parks). Apportioning criteria should be reviewed and periodically updated together with the risk estimations and in light of risk analyses of hazards not previously assessed.

5.4.6. The Issue of Low Probability – High Consequence Events

When developing societal risk evaluation criteria as proposed, the issue of high consequence events (large number of fatalities) having low, but existent, occurrence probabilities arises. Systems where the exposed population is small (of up to a few tens of even slightly over a hundred people) might be argued to show a balance between the low tolerated probabilities and the number of fatalities. However, for systems where large populations are exposed (densely populated developments where thousands of people can potentially be affected – such as communities downstream of dam facilities), it is difficult to decide on a tolerable threshold. It can be argued that on these circumstances the precautionary principle be applied. This principle states that where there are threats of serious consequences, all cost-effective measures to prevent them should be applied. In our context, this would imply that where the consequences are unknown but may be judged by some to be of catastrophic magnitude (large number of fatalities), it may be better to implement all known risk control measures or even to abandon the project rather than to accept the uncertain but potentially high risk (Skjong 2002). An example of the higher risk aversion towards large number of fatalities is the adoption of a vertical cut-off for tolerable risk thresholds when assessing the risk to developments in landslide prone areas in Hong Kong (Leroi *et al*. 2005).

5.5. EXAMPLE OF RISK EVALUATION CRITERIA DEVELOPMENT FOR RESIDENTIAL AREAS IN MOUNTAINOUS TERRAIN

Figure 5-3 presents a schematic illustration of existing and proposed residential areas in mountainous terrain. For simplicity, only three dangers are highlighted in this figure: Natural slope instabilities (deep seated slides, shallow slides, snow avalanches, rock falls, rock avalanches), river flooding, and periodic debris flows that shape the observed depositional fan. This section presents an example of developing proposed risk evaluation thresholds for developments in mountainous areas in Canada. It is mainly focused on slope instability related hazards, but its application can extend to all other hazards recognized in such areas.

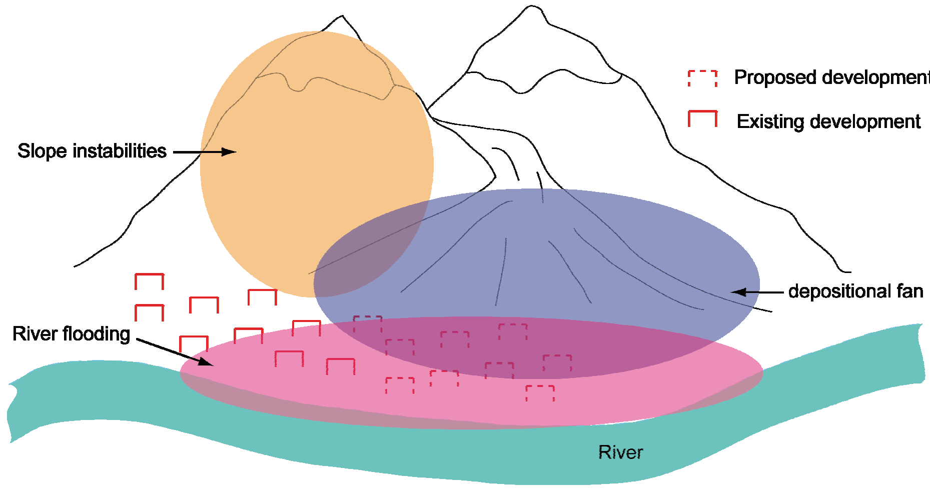


Figure 5-3 Schematic illustration of existing and proposed residential areas in mountainous terrain highlighting some potential ground hazards.

Porter *et al*. (2009) describe a case in the District of North Vancouver (DNV) where risk to life evaluation criterion was adopted as part of a quantitative risk assessment for developments in landslide prone areas. The Hong Kong evaluation criterion (ERM 1998) was selected on the basis of having a similar legal system (Common Law Legal System inherited from the United Kingdom) and to be developed for a similar context. The criterion developed in Hong Kong was based on previous studies for diverse industries within and outside the region (Dam management, transportation, nuclear power plants). Details on the definition of the criterion are presented in ERM (1998, 1999). Another good source detailing the concepts behind the definition of risk to public evaluation criteria in a similar context can be found in HSE (1998). The decision of adopting the Hong Kong criterion was supported by consultant's recommendations and informal feedback from the public. Porter *et al*. (2009) postulated that the Hong Kong tolerance criterion might be appropriate for application in Canada.

5.5.1. Individual Risk Criterion

Figure 5-4 presents the Hong Kong tolerable individual risk thresholds. The criterion allows for risks associated with existing situations to be one order of magnitude higher than for new situations. Figure 5-4 also shows the thresholds adopted by other organizations (HSE, ANCOLD and AGS). It is not surprising the risk thresholds are common between different organizations, which typically adopt the considerations and the methodology followed by the HSE (1988). Definition of the Hong Kong criterion (similar to the HSE criteria) mainly relied on assessing how common risks to the exposed population are related to the population's background risk (age standardized death probability by all causes).

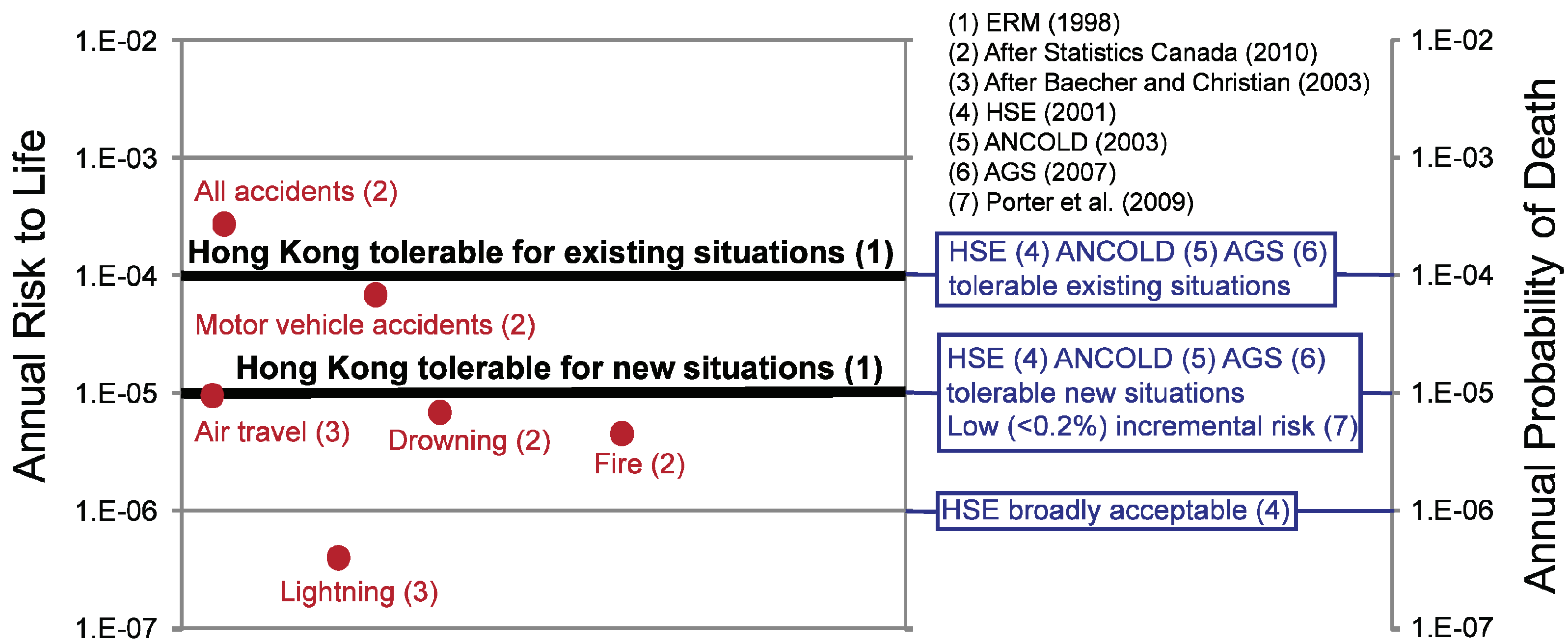


Figure 5-4 Individual risk evaluation criteria commonly cited in the geotechnical literature and common risks in North America.

The Hong Kong criterion is then compared to risks posed by activities common for the Canadian population in order to assess its applicability to the Canadian context (Figure 5-4). The tolerable risk threshold for new situations appear to be in the order of those risks imposed by air travel and drowning, and about an order of magnitude lower than for motor vehicle accidents. It is considered the population is willing to tolerate these risks related to transportation (for the case of air travel and motor vehicles) and their interaction with water bodies (for recreation purposes in pools and lakes). Risk tolerance for new situations in Canada can then be proposed to be around the same order of magnitude, which supports the adoption of the Hong Kong criterion. The threshold is further supported by the estimation of Porter *et al*. (2009) that this risk value (1E-5) corresponds to less than 0.2% incremental risk, which can be considered low. For existing situations, a proposed threshold value one order of magnitude higher would be about the risk of death due to motor vehicle accidents and lower than the risk of death due to all accidents.

It is believed the most important step in developing the risk criteria is its validation by the public exposed. In the case of the DNV described in Porter *et al*. (2009), a public task force convened by the DNV supported the adoption of the Hong Kong criterion based on a number of public meetings and public surveys.

Regarding acceptability thresholds, the HSE adopts a value of 1E-6, which for the Canadian context is about the same order of magnitude than events considered extremely rare (such as death by lightning in Figure 5-4). It also represents an increase of less than 0.02% in the standardized risk of death for the population. However, deciding if an acceptable threshold is to be adopted below which the ALARP principle is not mandatory, is responsibility of owners and regulator and should be done in consultation with the exposed population.

5.5.2. Societal Risk Criterion

Figure 5-5 presents the Hong Kong societal risk criterion (a) and criteria adopted by other organizations (U.S. Bureau of Reclamation, ANCOLD and HSE) (b). Unlike the individual criteria, the societal criteria vary among these organizations. This corresponds to differences in the type and scale of the hazards being evaluated and the number of people exposed. The Hong Kong criterion was chosen given the similar hazard context for which it was proposed (development is landslide prone areas).

Two court decisions published in the geotechnical literature where chosen to assess the applicability of the Hong Kong criterion. Both decisions implied the risks to the public were considered intolerable. The risk values were estimated after the decisions were made. The first case corresponds to rock fall hazards along a highway (Bunce *et al*. 1997) where a rock fall impacted a vehicle and killed one person. The second case corresponds to a proposed development in the path of a potential debris flow (Porter and Morgenstern 2012). Both estimated risks are plotted in Figure 5-5 and lie above the tolerable threshold line.

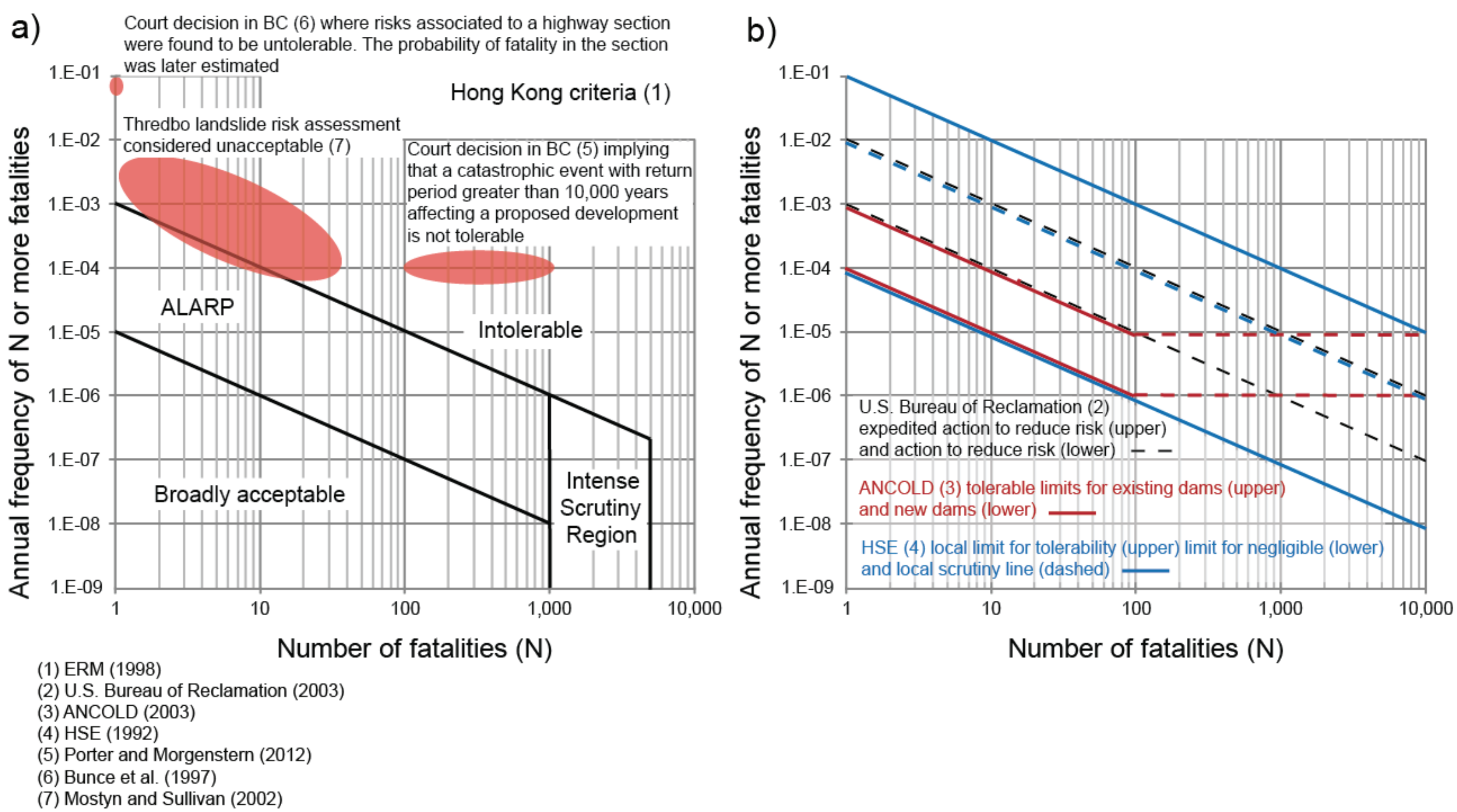


Figure 5-5 Societal risk evaluation criteria. a) Criterion proposed in Hong Kong for developments in landslide prone areas and cases where risks were found to be intolerable. b) Criteria commonly cited in the literature.

The Thredbo landslide in Australia (Mostyn and Sullivan 2002) is also plotted in this figure given the similar social and economic context. This suggests that the Hong Kong tolerable threshold might be applicable in the Canadian context for new developments. It also suggests that increasing the threshold one order of magnitude might not be applicable for existing developments, although more published cases should be analyzed.

The cut-off values adopted in Hong Kong for the area of intense scrutiny (between 1000 and 5000 fatalities) corresponds to a local policy, and its adoption needs to be based on political and social considerations. Again, here the involvement of regulator, consultant and the public is of critical importance. The same applies to the acceptability threshold.

5.5.3. Apportioning the Risk Criterion

The Hong Kong criterion was defined for a given areal extent, or "Consultation Zone"(ERM 1998). Porter *et al*. (2009) proposed a definition of the consultation zone for the Canadian context as: "The Consultation Zone shall include all proposed and existing development in a zone defined by the approving authority that contains the largest credible area affected by landslides, and where fatalities arising from one or more concurrent landslides would be viewed as a single catastrophic loss". This implies that the criterion is applicable to the area of influence of the hazard, and doesn't need to be scaled for the size of the Consultation Zone.

However, evaluation thresholds are applicable when all risks posed are integrated. If other hazards are considered negligible when compared to landslide hazards, the criterion described can be readily applicable to evaluate landslide related risks in the area. If the risk analysis is comprehensive regarding all hazards, the overall risk can also be readily assessed against the criterion. It is common for risk analyses to focus on one or a few particular hazards. The existence of other potential hazards needs to be considered when adopting threshold values for risk evaluation. Qualitative or relative risk assessments can shown to be useful to apportion the risk thresholds among hazards. If this information is lacking, a conservative approach can be to scale the thresholds to one order of magnitude lower, as long as no other hazard is deemed to pose higher risks than the one being evaluated.

5.6. EXAMPLE OF RISK EVALUATION CRITERIA DEVELOPMENT FOR RAILWAY CREW MEMBERS

According to WorkSafeBC (2009), transportation and related services is one of the high-risk economic activities in the province of British Columbia. Within the railway industry, train crew members and maintenance-of-way (MOW) personnel are the most exposed to operation hazards, which include a variety of ground-hazards such as rock falls, embankment settlements, river erosion, and rock and soil slides. The following section deals with a simplified example of how to develop risk evaluation criteria for railway crew members in Canada.

5.6.1. Corporate Individual Risk Criterion

5.6.1.1. Acceptable Risk Threshold

Proposing a threshold for risk acceptability involves answering the question of how much risk increment we are willing to accept. In this example an acceptability policy of zero risk increase is adopted. This does not mean that the risks associated with the activity are nil, but that these are not greater than the risks associated with avoiding the activity.

Risks to the average Canadian population (including crew members) arise from potential accidents and from several other causes (diseases, self-harm, crime). As long as the accidental risks during crew members working hours do not exceed the average accidental risks, it can be considered that the activity does not impose an increase on the individual's risk.

It is assumed the crew member (and MOW personnel) age is between 20 and 49 years. Figure 5-6 shows the annual life loss probability for the average Canadian resident within that same age group.

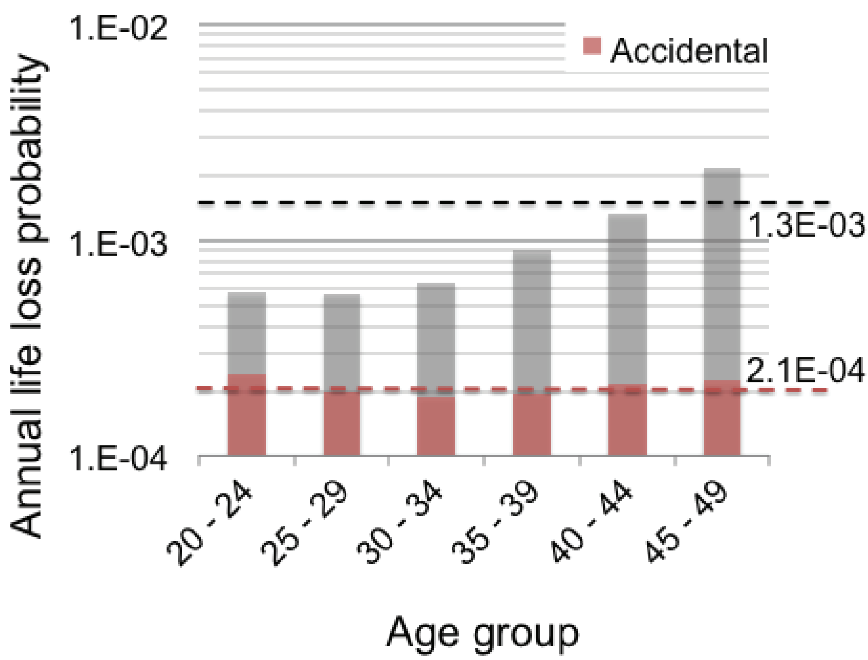


Figure 5-6 Annual life loss probability per age group based on the Canadian population mortality rates 2007 (Statistics Canada 2010).

For this age group, the average risk increment caused by work and non-work accidents is about 2.1 x 10^{-4}. Of course, this accident-related increment in risk is distributed throughout the day. In order to estimate the incremental risk corresponding to the period a crew member spends working, the following assumptions were made:

- Three periods of time where distinguished: working, sleeping and other, which would include common activities for the average Canadian resident,

- The average railway crew member and MOW personnel spend about 30% of the time working each year,
- The average railway crew member and MOW personnel spend 30% of the time sleeping (7 to 8 hours a day in average),
- Accident-related increase in risk while sleeping is considered low when compared to other activities, and;
- All accidents during the working period are considered work-related.

Table 5-1 shows the distributed accident-related risks for the average crew members and MOW personnel following the assumptions presented. Table 5-1 suggests that accident-related risks during working hours are about 6.3 x 10^{-5} per year. accounting for the errors from the assumptions adopted and the fact that not all accidents during the working period are work-related, an acceptable individual risk threshold of 10^{-5} can be proposed.

	Activity			All
	Working	Sleeping	Other	
% Time	30%	30%	40%	100%
Accident-related risk	6.3E-5	6.3E-5	8.4E-5	2.1E-4

Table 5-1 Accident-related risks for railway crew members and MOW personnel distributed throughout the day.

5.6.1.2. Tolerable Risk Threshold

Proposing tolerable risk thresholds requires understanding the increase in risk the workers are willing to tolerate in exchange for the benefits of the activity. Depending on the experience of the crew member or MOW personnel, it can be argued that each individual has a perception of the risk level associated with the activity. This perception is based on past experiences, shared experiences, and safety training. It is also argued that if the activity continues, and the worker has a perception of the associated risk; the average worker is tolerating the risk increase posed by the activity.

Work-related accident statistics can aid in quantifying activity-associated risks. These statistics for the province of British Columbia (WorkSafeBC 2009) were used to estimate the workers annual probability of death for four high-risk sub sectors of the economy. Table 5-2 presents the analysis. Also shown is the analysis for a low-risk sector as a reference. From these statistics, and considering that the estimates include workers exposed to variable levels of risk; a tolerable individual risk threshold of 1 x 10^{-3} can be proposed.

Sub Sector	Fatalities per year (a)	Claims per year (b)	Fatalities per Claims (c)	Injury Rate (d)	Annual Probability of death
Transportation and related services	26.6	4,098	0.65%	5.8%	3.77E-4
Construction	33.6	8,759	0.38%	5.9%	2.24E-4
Forestry	13.8	720	1.9%	6%	1.14E-3
Oil and gas or mineral resources	7.4	388	1.9%	2%	3.80E-4
Business services	1.2	711	0.17%	0% (less than 0.5%)	< 8.50E-6

a) Average number of accepted fatal claims per year between 2005 and 2009

b) Average number of accepted claims per year (short-term, long-term and fatal claims) for 2008 and 2009

c) ratio of a) respect to b)

d) Average number of claims (short-term, long-term and fatal claims) per 100 workers employed all year (per 100 person-years of employment)

Annual probability of death is estimated as c) x d) obtaining average number of fatalities per number of workers per year. Assumed to be a measure of likelihood of work related death of an average worker in one year.

Table 5-2 Employee annual death probability by sub sector estimated from work-related accident statistics by WorkSafeBC (2009).

5.6.2. Individual Risk Criterion Apportioned to a Specific Section

Apportioning of individual risk criterion to a specific section of the railway needs to consider the length of the section with respect to the overall extent of the system analyzed. In the case of crew members or MOW personnel, the extent of the overall system will be the length of the railway corridor they are assigned to.

For illustrative purposes, it is assumed that crew members are assigned to a corridor which is 300 miles in length. A critical section, 5 miles in length, is selected for a quantitative risk assessment given its long history of ground hazards. Assessment of the risk associated with rock falls is set as the main objective. Adopted individual risk evaluation criterion for the 5 miles should reflect the corporate individual risk thresholds as previously proposed. Even apportioning of the individual risk can then be obtained simply by multiplying the selected threshold times the ratio 5 miles / 300 miles. However, the fact that the 5 miles were selected for a detailed risk analysis over the other 295 miles is indicative of the greater risks posed by this smaller section. Even criterion along the entire corridor could lead to overall risk thresholds that are overly conservative.

Properly weighted apportioning would require knowledge of the risk levels along the 300 miles. Time and costs associated with quantitative analyses covering the entire corridor would make this requirement prohibitive. Qualitative assessments are a means to obtain relative risk levels along the corridor on which to base the weighted apportioning. When lacking these assessments, an experience-based estimation of the relative risk of the 5 miles with respect to the other 295 miles is needed. This implies that unless the risks are assessed along the entire corridor, significant judgment input is required for the adoption of weighted apportioning. However, decisions on weights can be facilitated by the absence of the hazard along certain sections (no recorded rock fall events, no perceived river erosion). For illustration purposes it will be assumed that the hazardous miles are about 100 times riskier than the miles of the rest of the corridor. The risk thresholds can then be estimated as:

$$(\text{Lower risk miles } x\ t) + \big(\text{higher risk miles } x\ (\text{weight } x\ t)\big) = T$$

where *t* is the apportioned risk threshold for each of the lower risk miles and T is the proposed corporate individual risk threshold. For the example the expression becomes:

$$295\ t + 5\ (100\ t) = T$$

The apportioned risk threshold for the 5 miles being analysed is given by $5\ x\ (100\ x\ t)$. These apportioned criterion can be applied to risk values that considered all potential hazards in their calculation. It is often the case that hazards considered are those believed to pose the higher risks, while other hazards through the section are considered negligible. In this situation, the apportioned criterion can still be representative of the corporate risk thresholds. However, it can be the case that assessments are focused on a particular hazard type and other potential hazards of a different nature are not analyzed. As an example, assessments focused on risks posed by slope instabilities should consider the potential for other hazards such as flooding or ballast fouling before adopting the apportioned criterion. Table 5-3 shows the individual risk criterion for railway crew members and MOW personnel in Canada as proposed in this example for the 5 hazardous miles. To account for other potential hazards, a simple approach of rounding down to the nearest order of magnitude was adopted. That would imply that it is believed other hazards could pose up to 5 times the risks estimated for the hazard analyzed.

	Corporate threshold	Apportioned non-weighted	Apportioned weighted	Apportioned weighted and considering other hazards
Acceptable	1E-05	1.7E-07	6.3E-06	1E-06
Tolerable	1E-03	1.7E-05	6.3E-04	1E-04

Table 5-3 Example individual risk evaluation thresholds proposed for train crew members and MOW personnel in Canada. Corporate threshold corresponds to the overall increase in risk for each individual. Other values are apportioned to the hazardous 5 miles along a 300 mile corridor.

5.6.3. Corporate Total Risk Criterion

The number of exposed crew members and MOW personnel at any given time is rarely above 3. Societal risk criteria would not be applicable to single events, as the potential consequences will have a maximum limit of three fatalities. However, total risk criteria can be thought of in the context of the overall business, where the corporate risk management policy requires limiting the number of annual fatalities for all potential events.

One option for a proposed risk threshold consists on evaluating a single measure of risk that would reflect the whole operation. The risk measure selected in this example is the overall probability of one or more fatalities. The example is developed considering that the railway employs up to 10,000 crew members and MOW personnel across Canada. Only the tolerance threshold will be proposed in this example.

Going back to Table 5-2, it is observed that for the Province of British Columbia (WorkSafeBC 2009) high risk jobs have a fatality rate (fatalities per individual working in the industry) between 2.2 x 10^{-4} and 1.1 x 10^{-3}. Again, it is argued that if the activity continues, and the worker has a perception of the associated risks; the average worker is tolerating the risk increase posed by the activity. It can also be argued that the public tolerate these fatality frequencies in exchange for the benefits related to these industries (although the forestry industry might be somewhere close to society's tolerable threshold). Considering that these statistics include workers exposed to higher risks than other workers within the same industry, and following an assumed policy of risk reduction; a fatality rate of 10^{-4} is selected as the company tolerance target for further calculations. The corporate total risk threshold can then be estimated using the Binomial Theorem (Bunce *et al*. 1997) as:

$$1 - (1 - f)^n$$

where f is the selected fatality rate and n is the number of employed crew members and MOW personnel. The corporate total risk tolerance threshold for a selected fatality rate of 10^{-4} and 10,000 employees would then be 0.63. Note that this corresponds to an annual frequency of 1 fatality. Even when this threshold could seem high if not familiarized with mortality statistics, it needs to be remembered that it actually reflects a low fatality ratio for the industry.

The shortcoming of using the probability of one or more fatalities at a corporate scale is that assessment of risk requires risk integration over all the track mileage and over all existing hazards. Even when this is ideal, it is currently not practical. Another option is to take the fatality rate of 10^{-4} as the corporate total risk tolerance threshold. This fatality rate will then be used for apportioning the tolerance threshold for site-specific assessments, based on the number of people exposed.

It can be argued that a tolerance threshold based on a fatality rate of 10^{-4} is not compatible with an individual risk tolerance threshold of 10^{-3}. However, the individual criterion is applied to the employees at most risk and reflects the individual willingness to perform a hazardous job in exchange for the benefit gained, while the total criteria reflects overall safety goals for the operation.

5.6.4. Corporate Total Risk Criterion Apportioned to a Specific Section

Apportioning of the corporate total risk tolerable threshold depends on the length of the section being analyzed with respect to the corridor length, and the number of employees assigned to that specific corridor. The same 5 miles within the 300-mile corridor will be assessed as for the individual risk apportioning. It is assumed there are 130 crew members and MOW personnel assigned to the 300-mile corridor. The tolerable threshold apportioned for the 300 mile corridor can then be estimated in a similar way as for the corporate threshold. For the adopted fatality rate of 10^{-4} and 130 personnel assigned to the corridor:

$$1 - (1 - (10^{-4}))^{130} = 0.013$$

This corridor's total risk tolerable threshold of 0.013 can then be apportioned to the 5 miles being analyzed in a similar manner as for the individual risk criterion. For a non-weighted apportioning:

$$\frac{5\,miles}{300\,miles}\ x\ 0.013 = 2.2\ x\ 10^{-4}$$

For a weighted apportioning assuming the 5 miles analyzed are about 100 times more risky than the other 295 miles:

$$295\,t + 5(100\,t) = 0.013$$

$$t = 1.6\ x\ 10^{-5}$$

$$\text{for the 5 miles} = 5(100\,t) = 8.1\ x\ 10^{-3}$$

If all potential hazards are being considered, these values can then be adopted for assessment. However, if only one hazard is being assessed, the threshold value should be reduced to account for the presence of other hazards. If the analyzed hazard is considered to be critical, being the other potential hazards of much lesser concern, the hazard specific tolerance threshold for the 5 miles could be set as half the calculated value, or 4 x 10^{-3} for the weighted apportioning. If the risks related to the other hazards are not well understood, a proper risk analysis of these hazards should be carried out. An interim hazard specific tolerance threshold could then be set by rounding down the previously calculated value to the nearest order of magnitude, or 10^{-3} for the weighted apportioning.

5.7. CONCLUSION

Quantitative risk assessments are becoming common practice for projects with high risks associated with them. A critical step in the risk assessment process is the adoption of risk evaluation criteria. Because of the diverse contexts for which previously defined criteria were proposed, different regions should derive their own criterion or perform an assessment of the applicability of any criteria to be adopted. As such, development of these criteria becomes necessary at a regional, industry, client, and even case specific scales.

The paper proposes a framework for the development of risk-to-life evaluation criteria. The framework is developed considering the main aspects involved (system characteristics, the socio-economic, cultural and political context) and the principles for developing the criteria. The framework is linked to the risk management process at its initial steps, ensuring the estimated risks and the defined thresholds are compatible.

The proposed framework basically consists of defining absolute risk thresholds and making a decision on the application of the ALARP principle. This structure was adopted to keep consistency with common practice within and outside the geotechnical community, and because it has shown its adequacy for risk communication. Methods for the development of risk acceptability and tolerance thresholds are also presented.

It is has become common practice to adopt the application of the ALARP principle for risk evaluations. This chapter presents two options for the evaluation of the ALARP principle besides the use of cost-benefit analyses. However, evaluating if the ALARP principle is being met is responsibility of owners and regulators, and the risk analyst can only provide parameter estimations.

Quantitative risk assessments are rarely comprehensive over the entire system and for all hazards present. So, methods for risk apportioning are discussed and treated in some detail in the examples presented.

Two simplified examples are presented. The first deals with existing and proposed development in landslide prone areas. The proposed risk evaluation criterion focuses on regulating the tolerable increase in risk for the population exposed. The second example deals with railway operations in the Canadian Cordillera and it focuses on regulating risks for freight train crew members. The objective of presenting these two cases is to highlight how different contexts lead to different reasoning when developing the evaluation criteria. The examples mainly discuss the use of some methods to define proposed risk threshold values and the apportioning of these to the problem being analysed.

Establishing risk evaluation criteria is not an easy task. The defined criteria, as shown by the examples, are highly dependent on the system being analysed and its context. This implies that the development of the risk evaluation criteria should be an integral part of the risk management framework, where not only the risk analyst takes part, but the regulators, clients and exposed population participate in establishing the criteria.

CHAPTER 6: DISCUSSION OF MAJOR FINDINGS

6.1. RESEARCH THROUGH CASE STUDIES

Part of the research was developed through case studies. This allowed for the challenges of performing a quantitative risk assessment (QRA) to be highlighted on real projects.

QRA of Miles 2 through 15 of the CP Cascade Subdivision

This section of the Canadian Pacific Railway (CP) Cascade subdivision is known for its high rock fall and rock slide frequency. These events can potentially block the track or hit a moving train, which could lead to a derailment. The assessment focused on the risk to life for freight train crew members due to rock falls and rock slides along this railway section.

Chapter 3 presents the process model and event tree developed to estimate the cut slopes related risks along this railway section. The details on how of the model variables were populated are also discussed. Results are then presented and assessed against selected criteria. For this particular case, two risk models were developed. The first model selected representative values of the input parameters, which lead to estimating a representative value of the risk level. In the second model, probability density functions (PDF) were defined for the input variables. A Monte Carlo simulation technique was applied to the model to obtain the probability distribution of the estimated risk level.

QRA of the Checkerboard Creek Rock Slope

The Checkerboard Creek rock slope is located 1.5 km upstream of the Revelstoke Dam, on the eastern slope of the Columbia River Valley. The importance of the Checkerboard Creek rock slope stability conditions is related to its location within the Revelstoke Dam reservoir, and to a lesser extent the existence of a secondary highway along its toe. A potential slope failure generating an impulse wave within the reservoir could compromise the dam structure and potentially flood the town of Revelstoke.

The slope under study is approximately 260 m high, with an overall slope angle about 30 degrees, being steeper at the lower area (45 degrees) and with a 50 to 60 degree slope cut at the toe. Chapter 4 present details on the slope geometry, geology, deformation mechanism, monitoring trends, deformation pattern, and models built to simulate and predict the slope behaviour.

In this case, failure volume scenarios were defined based on the known information about the slope (boundaries, geology, monitoring and numerical models). A qualitative assessment is then presented through the development of a Failure Modes and Effect Analysis (FMEA). It is proposed this step prior to a quantitative analysis will aid in achieving a comprehensive treatment of all potential hazards and their direct and indirect consequences. The subsequent analysis is concerned with the risk to life for the population at Revelstoke.

A process model is defined to estimate the occurrence probability of each failure scenario and another model is defined to estimate the consequences, given failure occurs. The input variables for the occurrence probability model were populated with PDFs based on elicited subjective probabilities. The model to estimate the consequences was populated partially with PDFs, with the exception of the earthfill dam breach probability and the flood levels given an event occurs. Population of the model variables and the justification for them are discussed in detail in Chapter 4.

Consequences are estimated as number of fatalities (N) against cumulative probability (F) of N or more fatalities through a Monte Carlo simulation routine. The variability in the slope failure probability is then used to estimate the ranges in societal risk levels and presented in a F-N plot. These risk levels are then evaluated against selected criteria.

This case history is further used to illustrate a methodology to combine quantitative risk assessments with early warning systems and the Observational Method for risk management of low frequency - high consequence events.

6.2. QUANTITATIVE RISK ANALYSIS AS A PROCESS

The events leading to a loss were modelled as a process. This study focuses on the loss of life of the population exposed to slope hazards. Each step within a process is either an event (such as a train derailment or a wave overtopping a dam) or a state (such as train speed). Each step can then be characterized by its magnitude (failure volume, velocity, overtopping height) and a probability for each magnitude. Visualization and description of these models was aid through the development of Event Tree Analyses (ETA).

The high frequency of slope failures along miles 2 through 15 of CP's Cascade subdivision allowed for the hazard analysis (failure volume, frequency and location) to be treated with a frequency approach. An ETA was then defined, which starts with a slope failure, and develops into the event branches considered representative and that would lead to the loss of life. Details on each step of the model are presented in Chapter 3.

Estimating failure probabilities of the Checkerboard Creek rock slope cannot follow a frequency approach. In this case, two models were developed, one to estimate the occurrence probability of each failure volume scenario and one to estimate the consequences given a failure occurs. The thinking behind them and how they were populated are detailed in Chapter 4.

Disaggregating the process into logic steps, starting with a slope failure and leading to a loss, allow for a more comprehensive analysis of all variables affecting the outcome of a slope failure. The process of quantifying each step further highlights the significance of these variables in the overall outcome, and the areas where improved knowledge will lead to a reduction in the estimated risk uncertainties.

The areas showing most uncertainty were the derailment conditional probabilities for the CP's Cascade subdivision case study, and the slope failure probabilities and run out velocities for the Checkerboard Creek rock slope case study. These exemplify the difference between high frequency events, where failure can be treated with a frequency approach, and low frequency events, where the complexity of failure mechanisms and prediction of post failure behaviour can be challenging.

Having the process disaggregated in logical steps also allows for analyzing where and what kind of mitigation strategy can be most cost/effective. This is discussed later in this chapter.

6.3. ACCOUNTING FOR UNCERTAINTY IN THE INPUT VARIABLES AND RISK ESTIMATIONS THROUGH MONTE CARLO SIMULATIONS

Population of the input variables within the process models and ETAs developed to estimate the risk levels can be done by assigning a representative value to each variable (most likely values). However, the uncertainty involved in each step of the process will lead to a level of uncertainty associated with the estimated risks. Sensitivity analysis can be performed to gain some insight on the levels of uncertainty related to the outcome. In this study, the input variables were assigned probability distributions between ranges of values believed to be the most realistic. Even when there still are uncertainties associated with eliciting the upper and lower values of these ranges, these uncertainties are believed to be significantly lower. This method does not eliminate the uncertainties related to the model input variables, but provides a method to minimize them and assess them in a clear and logic framework.

Several methods can be adopted to account for uncertainty carried from the input variables into the estimated outcome (First Order Second-Moment approximation - Chapter 1). This study adopted Monte Carlo simulation techniques that allow for the outcome probability distribution to be estimated.

In a Monte Carlo simulation, a uniform probability distribution through more than one order of magnitude will tend to randomly select 10 times more samples from the higher magnitude than from the immediately lower magnitude. This was the case for the subjective probability ranges when estimating the failure probability of the Checkerboard Creek rock slope. A linear cumulative distribution in a semi-logarithmic scale was adopted within the probability ranges. This causes the simulation to select a similar number of samples from each order of magnitude. This approach is believed to better reflect the tendency of a group of experts when eliciting subjective probabilities covering over one or two orders of magnitude.

The outcome of the Monte Carlo simulation is the probability distribution of the estimated risk. This distribution likely covers more than two orders of magnitude, reflecting the uncertainties in the input variables. When plotted in a normal scale, this distribution is concentrated towards the lower values and show a long tail towards the higher values. However, when plotted in semi-logarithmic scale, the shape of the distribution can be better assessed. It is important to note that this approach treats the risk orders of magnitude as risk categories, minimizing the effect of the actual estimated values when calculating the point estimates. However, it is believed the approach is compatible with how probability is perceived at orders of magnitude below 10^{-1}, and compatible with how evaluation criteria is expressed. Further, it allows for a measure of the uncertainty in the estimated risks.

Evaluation of the individual risk to life in the case of CPs Cascade subdivision study section makes it amenable for presenting the results as a probabilistic distribution. The greater number of population exposed in the case study concerning a potential failure of the Checkerboard Creek rock slope shifts the focus of analysis towards evaluating societal risks. It is common practice to do so through development of F-N plots. The outcome of the Monte Carlo simulation was interpreted as areas corresponding to the estimated societal risk and their uncertainty and presented in the F-N plots.

6.4. DEVELOPING RISK TO LIFE EVALUATION CRITERIA

Given the differences in the nature of the hazards, and due to the diversity in social, economic and political contexts, risk evaluation criteria need to be developed at regional, industry and even case specific scales. Chapter 5 presents some of the main considerations for developing risk to life evaluation criteria and proposes a framework for defining these criteria. Two simple examples of the development of proposed risk evaluation thresholds are also presented. It is noted that the framework can also be used to assess the applicability of previously proposed criteria to a specific context.

6.4.1. Initial Considerations

It is not the intention of this thesis to define any particular risk evaluation criterion, which should remain responsibility of owners, regulators and governments. The risk analyst should be the individual most familiarized with the characteristics of the system being analyzed. As such, it is considered essential for the risk analyst to be involved in developing the criterion, and for a clear and auditable framework to be followed.

The proposed framework leaves the social aspects to be considered by the owner or regulator. To select the principles that will lead to a clear framework for risk criteria development, it is argued that 1) there is a need to regulate risks posed by natural and cut slopes in a sound, clear and consistent manner, 2) risk thresholds aid in evaluating the real urgency for mitigation strategies, risks deemed as not tolerable would require mitigation to maintain a minimum quality of life, and 3) uncertainties inherent to a system can be dealt with by assessment and management of its associated risks, thus maximizing the benefit for the parties involved. The following combination of principles is selected:

Absolute risk criterion: The risk criterion will be formulated as a maximum level of risk that should not be exceeded, without due regard to the cost and benefit associated with it.

The ALARP principle: Dictates that risks should be managed to be As Low As Reasonably Practicable (ALARP).

The principle of equivalency: Risk should be compared with known levels of risks for similar activities or systems that are widely regarded as acceptable or tolerable.

The accountability principle: It implies transparent and clearly defined criteria, which should be quantitative rather than qualitative and based on objective assessments.

The holistic principle: Decisions regarding safety on behalf of the public should be based on a holistic consideration of all risks and apply across the complete range of hazards. Given the difficulties and effort this would require, the principle will be applied at the scale of the system being analyzed and will require simplification (apportioning and scaling).

Principle of parsimony: Simpler risk acceptance criteria might be preferable to complex ones.

The framework considers proposing threshold values for acceptable and tolerable risks. Risks above the tolerable threshold are considered not tolerable and risk mitigation is mandatory. An ALARP region is placed below the tolerable threshold. If the owner / regulator decides on adopting an acceptability threshold (risks below this threshold need not further reduction), the ALARP region lower boundary will be determined by it. Else, the ALARP concept is applied to all risks lying below the tolerable threshold. The framework also considers risks to be assessed in terms of the individual risk (for the individual estimated to be at highest risk) and societal risk (through F-N plots).

6.4.2. Proposed Framework

The proposed framework is divided in three major stages according to the major participants that should lead the analyses. The first stage is considered to be carried out through a coordinated effort between the risk analyst and the owner / regulator, and needs to be consistent with the risk analysis scope of work. The population within any system can be grouped in three classes 1) Workers, which are people exposed to the hazard in exchange of economic or professional gain, 2) Users, which are people exposed to the hazard in exchange for a gain or to cover a need, and 3) Public, which are people exposed to the hazard not being aware of any direct benefit in exchange. This grouping is fundamentally about defining voluntary and involuntary risk levels.

The second stage should be carried out by the risk analyst given his acquired knowledge of the entire system. The steps in this stage need to be followed for each population type and for individual and societal risks, as applicable. The steps, assumptions, and principles behind the development of the proposed criteria need to be clearly stated, as they will be reviewed in the later stage by the owner / regulator and, ideally, the population exposed.

The last stage is responsibility of the owner / regulator. It consists on making final decisions and adjustments to the proposed criterion considering the social, economic, political and cultural context. The risk analyst acts only as a consultant and it is strongly recommended for the public to be involved in the decision-making process. The risk criterion should be reviewed and periodically updated in light of changes in both the hazard and the elements at risk.

6.4.2.1. Methods for Developing Proposed Risk Thresholds

Some of the methods currently available are 1) comparing against mortality statistics within the industry or for similar industries and activities, 2) comparing against natural hazard mortality, 3) comparing against common risks, 4) based on previous decisions in similar contexts, 5) comparing against existing criteria for similar contexts.

When developing proposed risk thresholds it is important to keep in mind the following considerations consistent with the principles previously discussed:

- Incremental risk associated with the system analyzed should not be significant when compared to the risks associated with everyday life, and whenever possible the ALARP principle be applied.
- Events with the potential to cause large number of fatalities should have low occurrence probabilities. This accounts for society's lower risk tolerance to large numbers of fatalities.
- Some populations will tolerate higher risks than others in relation to the required efforts to mitigate those risks, the benefit gained from their existence or the activity realized (workers in the mining industry as opposed to the public).
- Tolerable risks are higher for natural slopes than for those engineered or controlled (slope cuts, earth fill dams, or even for natural slopes that are known to be monitored).

6.4.2.2. Methods for Evaluation of the ALARP Principle

One method to evaluate if the ALARP principle is being met is through conventional cost-benefit analyses. This implies assigning an economic value to life, however the approach conflicts with ethical traditions.

A preferred method is the use of a cost-effectiveness analysis, which calculates a ratio of the cost of implementing risk reduction measures to the reduction in risk, thus avoiding putting an economic value to life. Two commonly used variants of the method are the adjusted cost-to-save-a-statistical-life (ACSSL) and the unadjusted cost-to-save-a-statistical-life (UCSSL), which are explained in Chapter 5.

Assessing if the ALARP principle has been satisfied, however, requires criteria regarding the CSSL values considered cost-effective. Deciding what is considered to be cost effective CSSL values is not simple matter, but it can show to be a strong tool in decision making between mitigation options against the no-action option.

6.4.2.3. Methods for Apportioning the Criteria

Risk criteria are often defined for a certain scale of the system. These criteria need to be adjusted to reflect the scale of the particular system evaluated, or have to be apportioned throughout the sub-systems for their individual evaluation (risks at a specific mileage deemed of high risk within a transportation corridor). If the criteria are not scaled or properly apportioned, higher risks than desired could end up being tolerated in some cases, or the criteria might show to be too conservative in others.

Apportioning of the criteria can be done assuming a uniform distribution of risks through all the components of the system (each mile of a transportation corridor poses the same level of risk, thus the criterion is equally apportioned throughout the corridor). Moreover, it can be assumed all hazards to pose similar risk levels, thus the criterion is equally apportioned for each hazard being analyzed. On the other hand, the hazard being analysed can be assumed to pose such level of risk that other hazard risk levels can be neglected. In this case the criterion can be entirely applicable to the hazard being analysed. The brief examples presented in Chapter 5 illustrate these concepts.

6.4.2.4. Low Probability - High Consequence Events

When developing societal risk evaluation criteria as proposed, the issue of high consequence events (large number of fatalities) having low, but existent, occurrence probabilities will arise. It is difficult to decide on a tolerable risk threshold for systems where large populations are exposed (densely populated developments where thousands of people can potentially be affected – such as communities downstream dam facilities). It can be argued that on these circumstances the precautionary principle be applied, however, the principle could eventually lead to a state of no further development of new projects, or abandon existing ones, with the associated overall risks to society. In Chapter 4 it is suggested that one way forward is the systematic application of QRA coupled with early warning systems and the Observational Method, as an aim to minimize the exposed population. This is summarized later in this chapter.

6.5. QRA AS A DECISION MAKING TOOL

Disaggregating the process into logic steps, starting with a slope failure and leading to a loss, allow for a more comprehensive analysis of all variables affecting the outcome of a slope failure. It also allows for analyzing where and what kind of mitigation strategy can be most cost/effective and thus optimize resource allocation.

Figure 6-1 shows part of the process model leading to a loss for CPs Cascade subdivision case study. In this figure different mitigation options are listed on the top corresponding to each of the highlighted areas. These areas correspond to those steps where the mitigation strategy directly affects the magnitude or probability of each step. Mitigation options being considered for risk reduction can then be related to particular steps of the quantitative analysis.

In this manner, the effectiveness of a mitigation approach can be translated to a reduction of the magnitude or probability in the corresponding step within the model. In turn, this will render a different risk level. The costs of the mitigation approach can be normalized by this change in risk level and an index be obtained. comparing the index for different mitigation options can then be used as a tool for decision making regarding risk mitigation options.

Figure 6-2 shows part of the process model leading to a loss for the Checkerboard Creek rock slope case study. In this context mitigation of risks focussing on reduction of the hazard levels (failure occurrence probability) can prove to be cost prohibitive and even technically challenging. Note however that drainage in slopes showing high pore water pressures as driver of the instability mechanism has been proven to be cost effective. When this is not the case, preferred options typically aim to reduce the consequences of failure.

In the case of a failure of the Checkerboard Creek rock slope, consequence reduction through management of the reservoir levels, enhancement of the dam robustness and flood control works can all reduce the risk levels. However, residual risks will still be present for a large population exposed. Monitoring and early warning systems, as risk mitigation option, aims to minimize this number of potential fatalities through timely evacuation.

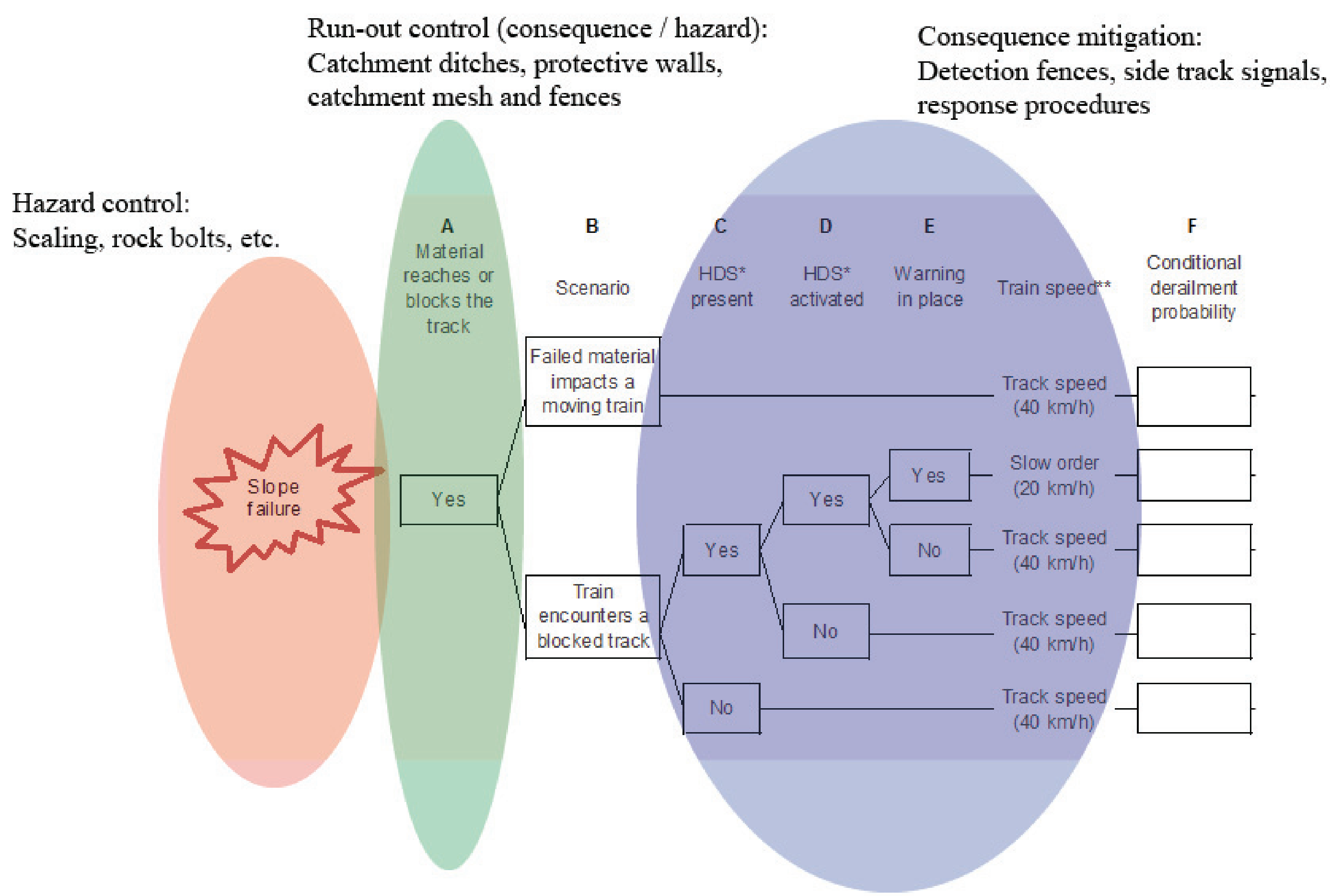


Figure 6-1 Mitigation strategies and their effect on different steps of the process leading to a loss - CPs Cascade subdivision case study.

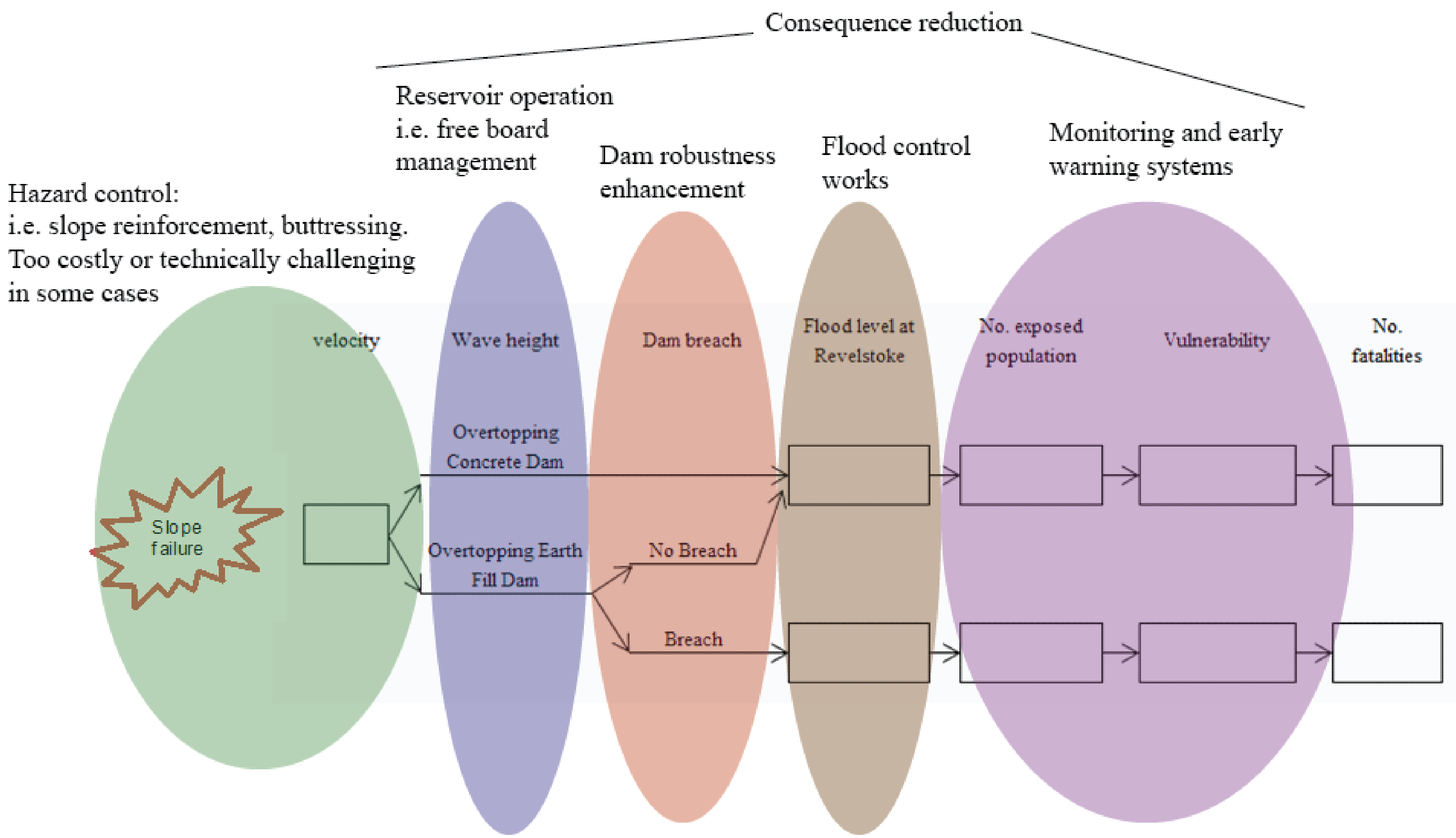


Figure 6-2 Mitigation strategies and their effect on different steps of the process leading to a loss - Checkerboard Creek rock slope case study.

6.6. COUPLING QRA, MONITORING, EARLY WARNING SYSTEMS AND THE OBSERVATIONAL METHOD

The QRA developed for the Checkerboard Creek case study indicated there is a residual (but real) probability of an event leading to over 7,000 fatalities. Focusing on reducing this residual risks to nil when dealing with large slope instabilities can be cost prohibitive and technically challenging. The issue of residual risk tolerance for catastrophic consequences is one subject that QRA and risk management haven't been able to fully resolve.

Coupling between QRA, early warning systems and the observational method seems to be one way forward to achieve a cost effective, robust and reliable risk management approach for large slopes. Chapter 5 presents an approach based on coupling these concepts. In this approach, QRAs are not only used to evaluate the current risk levels, but to measure its variability if changes in the slope behaviour and characteristics are noticed through monitoring. This requires the risk analysts to foresee these possible changes. This is associated with an increase in the amount of effort required for analysis, however, even if done in a simplified manner and through increased input of judgment, a better understanding of the new threats can be gained.

Early warning and emergency plans are linked to selected thresholds on the parameters being monitored. The approach, however, allows for these thresholds to be associated with changes in risk levels and can be selected on the basis of an improved understanding of these new risk levels.

In a risk management context, the Observational Method relates the adequacy of the risk mitigation strategy for the risk level estimated. This depends on the hazard levels related to the slope according to the monitoring information at the time. Foreseeing potential changes in the slope behaviour, noticeable through monitoring and related to a new risk level, would then lead to the application of previously assessed mitigation strategies that would take the risks to acceptable levels. As an extreme, the decision for evacuation of a sector of the population could be included as one of these strategies, related to monitoring results indicating a large scale failure might be imminent.

It is noted here that observation should not only apply to changes in the slope behaviour and conditions, but also changes in the characteristics of the elements at risk and the surrounding environment (population density, reservoir levels, new infrastructure, stabilization works, weather events and climate change).

Given the usually prohibitive costs related to risk mitigation strategies that minimize the probability of failure of large slopes, and the technical difficulties associated with them, monitoring and early warning systems are typically the options of choice. However, quantification of risk reduction due to monitoring and early warning systems is not an easy task. In controlled environments such as the open pit mining industry, where the number of exposed workers and equipment is limited and workers are required to follow safety instructions, early warning systems can effectively lead to timely evacuation and consequence reduction. When the population at risk is the general public, and for a large exposed population, factors such as an effective risk communication, the warning being issued efficiently, evacuation procedures being carried out as planned and the people response towards an imminent catastrophe (just to name a few) will play a major role in the effectiveness of the early warning system.

Approaches developed to estimate life losses related to flooding events consider the effects of warning times and evacuation success levels (See Chapter 4 for references). These, however, are associated with much uncertainty related to environmental conditions, public response, distance from the hazard and to a safe zone, resources available for evacuation, and knowledge of the evacuation routes and procedures. The success rate of early warnings need to be assessed for each specific case, and ideally calibrated by measuring the time and number of people evacuated during evacuation drills.

Regarding the adoption of early warning systems for risk mitigation, there is the risk associated with false warnings undermining the public trust in the system and dramatically lowering the evacuation success rate. Adequate and transparent risk communication, explaining the uncertainties inherent to the phenomenon, the risk levels and the potential for false indications of sudden failure should aid in reducing this risk.

CHAPTER 7: CONCLUSIONS AND FUTURE RESEARCH

Natural and engineered slopes pose potential hazards to the public, workers, infrastructure, economy and environment. Historically, management of risks associated with natural and cut slopes has been done through the adoption of several approaches. In most early cases, risk assessments were only implicit and subjective within the decision making process for slope management. A generic quantitative risk assessment (QRA) framework for natural and engineered slopes now exists. This is being widely adopted by the geotechnical community that has accepted QRA as main tool for slope risk management. It is clear that the limitations of the framework need to be addressed systematically in order to achieve its full potential and acceptance. The present research addresses some of these limitations.

Two case studies were presented in this research, selected to represent two opposite types of geotechnical problems. These are the Canadian Pacific Railway (CP) Cascade subdivision case study, representing high frequency - low consequence scenarios, and the Checkerboard Creek rock slope, representing low frequency - high consequence scenarios. The QRAs focused on particular hazards and particular elements at risk for each case, however they were comprehensive in the treatment of all factors affecting the estimated risks. These case studies show how QRAs can be developed in a comprehensive manner and yet readily applicable by the practitioner.

Conclusions particular to each of the two case studies presented can be found at the end of the corresponding Chapter (Chapter 3 for CP's Cascade subdivision and Chapter 4 for the Checkerboard Creek rock slope). This chapter is devoted to general conclusions that arise from the analysis of the entire research, which are linked to some needed future research.

7.1. PROCESS MODELLING FOR QRA

Development of process models, particularly with the aid of event tree analysis (ETA), allows for disaggregating the sequence of events leading to a loss into logic steps. Unlike other approaches, such as the use of safety factors, QRAs through process models permits a more comprehensive and clear analysis of the variables affecting the outcome of a slope failure. More importantly, it facilitates detailed review of the process, assumptions, and simplifications such that updates and improvements can be built on previously developed models.

The process of quantifying each step further highlights the significance of these variables in the overall outcome, and the areas where improved knowledge will lead to a reduction in the estimated risk uncertainties. This can help define future studies that would more efficiently reduce the levels of uncertainty and optimise resource allocation for risk mitigation purposes. In this regard, QRA analysis of the processes leading to a loss highlights the steps where mitigation measures result in larger reductions in risk. Moreover, it highlights the riskier scenarios in a quantitative manner.

Highlighting the areas were improved knowledge is needed also leads to the identification of those aspects that might need more attention from the risk management community and the research community in general. An example is found in Chapter 3 where the conditional probability of a derailment given the train impacts a blocked track is needed to estimate the risk. Another example is given in Chapter 4 where the Revelstoke earthfill dam robustness against impulse wave overtopping needed to be assessed.

The process models developed in this thesis for the two case studies also highlight the effort required to estimate the consequence magnitude and probability after a slope failure. In most analyses, the chain of events and varying scenarios in the processes leading to a loss after a slope fails makes the consequence analysis the most demanding part of a risk analysis. Moreover, the risk analyst has to evaluate how meaningful an estimated value of risk is when compared to the estimated consequences. As it is discussed in this thesis, care is needed when addressing situations where slope failure probabilities are small and consequences are large, rendering an overall low estimation of risk.

7.2. UNCERTAINTY IN THE ESTIMATED RISK

Different sources of uncertainty were discussed in Chapter 2. When a process model is developed to estimate the risk levels of a particular system (slope - elements at risk), uncertainty in the values of the input parameters is carried through the analysis and is reflected in uncertainty in the estimated risk. Some steps pose greater uncertainty than others, depending on the available information and the current knowledge of the subject, leading to the necessary input of expert opinion as opposed to based solely on observations or analyses.

The approach adopted in this research starts with the premise that the uncertainties related to defining a unique value for the input variables are significantly greater that eliciting the range of possible values the variables are believed to take. The uncertainty can be modeled further by defining a probability density function (PDF) for the elicited range of values. This allows for formal methods to be applied for estimating the uncertainty in the estimated risk, related to the input variable uncertainty. Moreover, it allows for updating of the input variables PDF in light of new information and knowledge.

It is noted here that uncertainty is still present about the upper and lower values of the elicited range of values for the input variables. Also, model uncertainties, biases and human uncertainties are not deal with by the approach, although model uncertainty should be limited given a proper elicitation method and peer review (as mentioned in Chapter 2).

With this in mind, QRA presents a way forward to deal with uncertainties in geotechnical practice in a systematic and clear manner, but future research is needed on how to assess the other types of uncertainty. A proposed next step is to evaluate which types of uncertainty are present for each step or level of the QRA process, and what is their significance. A methodology can then be proposed to deal with them in a systematic way. Given the difficulties in doing so, a qualitative approach seems suitable as a first attempt. Case studies should be used in order to assess its applicability in real situations.

7.3. EVALUATION AND VISUALIZATION OF RESULTS

Given the uncertainties in the input variables required to populate the process model for the estimation of risks, results usually cover more than two orders of magnitude. The estimated risk probability distribution then tends to be concentrated in the lower values and show a long tail towards the higher values. It is believed that each order of magnitude when estimating risks - with typical values lower than 10^{-2} - is perceived as a risk category, and the risk probability distribution plotted in natural scale does not properly represent this notion. The approach presented in this research calculates point estimates and plots the estimated risk probability distribution in semi-logarithmic scale. This also enhances the use of graphics for visualizing the results, which will aid in achieving an effective risk communication.

The approach for risk visualization was applied when evaluating individual risks, however the evaluation of societal risks is proposed to be done through the familiar F-N curve (number of fatalities plotted against their cumulative probability). The uncertainty in the estimated risks can then be presented as the areas where the risk value and uncertainty lies, rather than probability distributions of the results. In this regard, particular number of fatalities (N) can be chosen for detailed analysis and their probability distribution plotted as for the individual risks.

It is proposed future research should focus on applying this evaluation and visualization approach to a variety of case studies in order to test its robustness and find its practical weaknesses. This should also point the way forward for improving the approach.

7.4. RISK EVALUATION CRITERIA

A proposed methodology for the development of risk evaluation criteria is presented in Chapter 5. It is believed the risk analyst, having the most knowledge on the hazard and elements at risk, should be actively involved in the development of these criteria. Final decision, and liability, on the risk levels to be considered acceptable or tolerable should be responsibility of the local government, regulator agency and/or operator.

The methodology proposed is based on the current practice within the geotechnical community and other areas of engineering and sciences. The methodology clearly states the people responsible for each step, based on the technical and societal contents associated with each step. It also presents the tools available for defining the risk thresholds considered as acceptable or tolerable and methods available to assess if the ALARP (as low as reasonable practical) principle is being met.

It is now clear that the different contexts and the particularities of each case study demand risk criteria to be defined, almost (or always) for each particular case. This arises from the fact that different cases will require different scaling and apportioning. As such, even when risk criteria can be defined at an industry, provincial or company level, these criteria need to be properly modified for the scale of the case study, the relative risk posed by the hazard being analysed, and the number of hazards present. Chapter 5 presents examples of these.

When the estimated risk is presented as a probability distribution, as proposed in this research, it seems reasonable to add an extra component to the risk evaluation process. On one side, the significance of the model can be evaluated based on the amount of spread the model outcome shows. It is proposed that a strong model would have an outcome constrained within two or three orders of magnitude. This can be assessed through evaluation of point estimates when plotted in semi-logarithmic scale (less than 2 orders of magnitude between +/- 2 standard deviations from the estimated mean, or between the 5% and 95% percentiles, as calculated using the logarithm of the model outcomes). Models showing larger spreads might not be reliable and a review of the process model would be required as well as further studies to populate it.

On the other hand, the evaluated risk probability distribution can show point estimates below the adopted maximum thresholds, but a percent of results might lie above them. A limit for the percent of results above these threshold values would need to be set as criterion in such cases.

Future studies regarding the adoption of risk evaluation criteria should address several areas of study. Risk perception can significantly influence decisions when developing risk evaluation criteria. Development of these criteria need to be based on sound analysis rather than false amplification of the estimated risks or unrealistic safety expectations. In this regard, much research is necessary on risk communication and the fair distribution of risk and benefits from the system being analysed. Even when these studies seem to be the domain of social and political sciences, it is the risk analyst who has the detailed knowledge of the system, and should be actively involved.

7.5. LOW FREQUENCY, LARGE SCALE SLOPE INSTABILITIES ASSOCIATED WITH HIGH MAGNITUDE CONSEQUENCES

With the possible exception of reducing pore pressures along a basal shear surface, risk mitigation measures for large slopes aiming to increase their stability or protect the elements at risk are usually cost prohibitive and technically challenging. In an attempt to cope with these situations, monitoring and early warning systems are typically adopted. It can be argued that increasing the resources to mitigate these risks can lower them to negligible levels. However, in most cases there is still the potential for a residual probability for large numbers of fatalities to be present. This is one subject that QRA and risk management haven't been able to fully resolve.

Coupling between QRAs, early warning systems and the Observational Method seems to be one way forward to achieve a cost effective, robust and reliable risk management approach for large slopes. The objective of early warning systems is to minimize the population exposed, thus avoiding large numbers of fatalities. However, warning needs to be timely, reliable, and communicated effectively. Another shortcoming, as illustrated in Chapter 4, is the increase in the amount of analysis required to adopt this approach, when attempted in a quantitative manner. In this regard, it is proposed case studies be developed in the future coupling QRA with the Observational Method and early warning, but in a qualitative or semi-quantitative manner. Simpler cases can then adopt a quantitative approach as discussed in this research, with simplifications such as neglecting scenarios considered unrealistic.

When the estimated risks approach or exceed the adopted evaluation thresholds, it is necessary to quantify the reduction in risk after the early warning system is in place. When the population at risk is large, factors such as an effective risk communication, the warning being issued efficiently, evacuation procedures being carried out as planned and the people's responses towards an imminent catastrophe (just to name a few) play a major role in the effectiveness of the early warning system. The success rate of early warnings need to be assessed for each specific case, and ideally calibrated by measuring the time and number of people evacuated during evacuation drills. This is a topic that needs to be addressed in future research.

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