# Risk management of large rock slopes – state of practice

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ABSTRACT
Assessing the stability of rock slopes requires a through understanding of the failure mechanisms. For large natural rock slopes there is usually a limited amount of geological information and hence a complete understanding of the slope is often lacking. Risk assessment of such slopes can be used to augment traditional analyses and this may be carried out using a general framework for landslide risk management. The Checkerboard Creek rock slope, located 1.5 km upstream the Revelstoke dam, BC; has been widely studied and monitored since the late 70's. This case history is used to demonstrate the demands required for a quantitative risk assessment methodology for large natural rock slopes.

RÉSUMÉ
Évaluation de la stabilité d'un talus rocheux a besoin d'un grâce à la compréhension des mécanismes de défaillance. Pour de grandes pentes rocheuses naturelles il ya généralement une quantité limitée d'informations géologiques et, partant, une compréhension complète de la pente est souvent défaut. L'évaluation des risques de pistes peuvent être utilisées pour compléter les analyses traditionnelles, ce qui peut être effectuée en utilisant un cadre général pour la gestion des risques de glissements de terrain. La pente Checkerboard Creek, situé à 1,5 km en amont du barrage de Revelstoke, en Colombie-Britannique; a été largement étudiés et surveillés depuis la fin des années 70. Cette histoire de cas est utilisée pour démontrer les exigences requises pour une méthode d'évaluation quantitative des risques pour les grandes pentes rocheuses naturelles.

## 1 INTRODUCTION

Geotechnical engineering is fundamentally about managing risk. Morgenstern (1995) summarized risk assessment concepts using the framework for risk management adopted by the Canadian Standards Association (Figure 1).

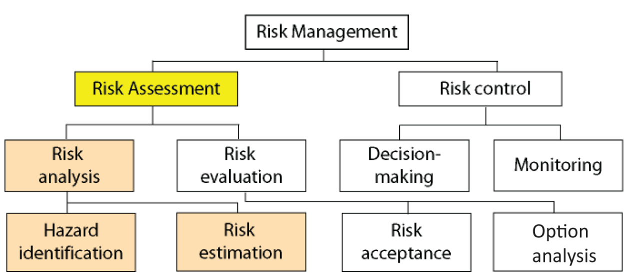


Figure 1: A framework for risk management (CSA, 1991).

Morgenstern noted that while quantitative risk analysis (QRA) is one aspect of the framework, qualitative risk assessment is also a valuable component of risk management. With risk defined as the combination of the probability of occurrence of an undesired event and the possible extent of the event's consequence, risk can, in principle, be calculated. The full potential of QRA is best met with the establishment of acceptable risk criteria. This is not an easy matter, particularly in geotechnical engineering. Relating consequences to cost/benefit analysis provides a simpler basis for evaluating acceptable risk. The link between risk and benefit must be balanced and within the context of geotechnical engineering, the risks are usually reduced to as low as reasonably achievable (ALARA) by the best practical means.

Assessing the stability of a large rock slopes is problematic. It requires the development of a comprehensive geological model and a through understanding of the failure mechanisms. For large natural rock slopes there is usually a limited amount of geological information and hence a complete understanding of the slope is often lacking. Risk analyses can be used to augment our traditional analyses for such slopes but this approach is not without its own challenges. Ho et al (2000) outlined the quantitative risk assessment strategy that has been implemented in Hong Kong to augment the conventional approach for certain classes of landslide problems. In 2000, the National Committee of the Australian Geomechanics Society released the "Landslide risk management concepts and guidelines" in an effort to establish a more formal framework for landslide risk management. This document was updated in 2007 in response to users' comments. As noted by Ho et al (2000) formal quantitative risk assessment procedures for landslides are seldom universally accepted and therefore are usually applied as pilot studies, or to tackle a site-specific problem. This reflects the fact that the adoption of quantitative risk assessment techniques in geotechnical engineering is still in the early stages of development as an emerging concept. Formal risk assessment procedures create a framework for a preliminary assessment of all slope hazards and focus engineering efforts and expenditures on the highest risk areas (e.g. Pine and Roberds, 2005).

The purpose of this paper is examine current approaches for assessing the risk for large natural rock slopes and to evaluate how these could be applied to the Checkerboard Creek slope, a large rock slope near Revelstoke BC, which is showing an annual deformation pattern.

## 2 RISK MANAGEMENT APPROACHES FOR LARGE ROCK SLOPES

The approaches for assessing the risks inherent in large rock slopes are varied. A simple approach relies only on past performance of similar structures. This implies that the risks are a minimum as long as the structure has the same or improved conditions as others showing acceptable performance. This approach has evolved into empirical design guidelines, widely used for the design of open pit slopes in the mining industry and road cuttings (Hoek and Bray 1981, Wyllie and Mah 2004, Haines and Terbrugge 1991). The limitations of this approach are well recognized and its use is often restricted to the initial stages of a risk assessment.

### 2.1 Factor of Safety Calculation as Risk Management Approach

The computed factor of safety (FS) based on traditional limit equilibrium methods and evaluated against some acceptability criteria, e.g., FS>1.3, constitute an indirect assessment of risk. Apablaza et al. (2000) used this approach for the Sur Sur Mine open pit slopes where, as for many other open pit slopes, the design was based on minimum factors of safety adopted for interramp and overall slopes. Hoek (2007) also used the factor of safety approach to assess the risks for the Sau Mau Ping Road cut in Hong Kong. Hoek evaluated the factor of safety for the in situ conditions and for several stabilization options, and compared those results against the required minimum values. This approach provides a basis for evaluating risk associated with various mitigation options. Hoek evaluated the mitigation options by comparing how much the factor of safety increased with respect to the no-mitigation factor of safety. This "Relative Factor of Safety" approach has also been used to evaluate stabilization options for extremely large rock slopes (Hoek 1991).

### 2.2 Probability of Failure Calculation as Risk Management Approach

The use of probabilistic slope stability analysis started in the 70's to account for the uncertainties introduced when analyzing slopes. As pointed out by Morgenstern (2000), "Uncertainty is chronic in geotechnical practice and quantitative prediction of behaviour, even under ideal circumstances, is unreliable". It was recognized that higher factors of safety do not necessarily mean safer slopes, if the uncertainties in the calculations are also higher; and that while the probability of failure is proportional to the likelihood of failure, the same is not true for the factor of safety (Tapia et al. 2007). The calculation of the probability of failure and its evaluation against some criteria for tolerable values, are being widely adopted by the mining industry as a way of limiting the risks related to open pit slope design and optimization (Loubser 1994, Yang et al. 1999, Tapia et al. 2007, Mathis 2007).

### 2.3 Slope Deformation Monitoring as Risk Management Approach

It has been documented that slope failures in soil and rock generally occur after a period of increasing rate of movement (Martin 1993, Leroueil 2001). Extensive monitoring of open pit slopes has lead to the recognition of different phases of the slope deformation response to changes in stresses (Zavodni 2000). Slope movement monitoring has now become standard practice for open pit and natural slopes, and it is used as an indicator that failure is about to occur when compared to some threshold criteria based on experience (Hungr et al. 2005). Methodologies and criteria have been proposed to estimate the time to failure which minimizes the failure consequences, e.g., see Rose & Hungr, (2007), Zavodni (2000) and Fukuzono (1985).

Continuous monitoring and review of open pit slope deformations allow for steeper slopes and minimizes waste rock. When experienced based thresholds are exceeded, machinery and personnel are evacuated from the area before failure occurs. Zai-Nan and Guo-Zheng (1994) report on the case of the East Open Pit in Daye, China; where a slope failure was successfully predicted by continuous slope deformation monitoring, allowing operations to continue until the risk for equipment and workers was considered above tolerable levels and evacuation done before the localized failure occurred. The costs associated with intensively monitored open pit slopes which failure is predicted by changes in its deformation characteristics, are usually lower than the costs associated with operating using flatter slopes.

### 2.4 General Risk Management Framework

Various methods of risk management have been proposed in the last three decades that 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; as an iterative and continuous process. The details for each step of the process and the different methods and tools available for estimating the values required at each step have been described by Fell et al. 2005, Crozier and Glade 2004; Lee and Jones 2004; and hence those details are not described in this paper. These approaches require that the risk values obtained have to be evaluated against some criteria adopted, and management decisions and mitigation strategies must be based on that evaluation. A significant step forward compared to other approaches is that a quantitative magnitude of risk is estimated by systematically "weighting" the likelihood of each possible event by the magnitude and likelihood of its consequences. These risk values can then be compared to acceptable risk criteria posed by different industries (i.e. aviation, nuclear power generation, dam safety, etc) and also to the risk from natural hazards (i.e. earthquakes, Hurricanes). Examples of the adoption of this approach are the Aknes Slide in Norway (Lacasse 2008), The Rosone Landslide in Italy (Amatruda et al. 2004) and the Sedrun Landslide in Switzerland (Bonnard et al. 2004).

## 3 CHECKERBOARD CREEK ROCK SLOPE

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 that lacked a through going basal shear zone dipping out of slope. 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 2). 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.

### 3.1 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 2). 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. 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 2 shows the location and approximate boundaries of the Checkerboard Creek rock slope.

### 3.2 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,

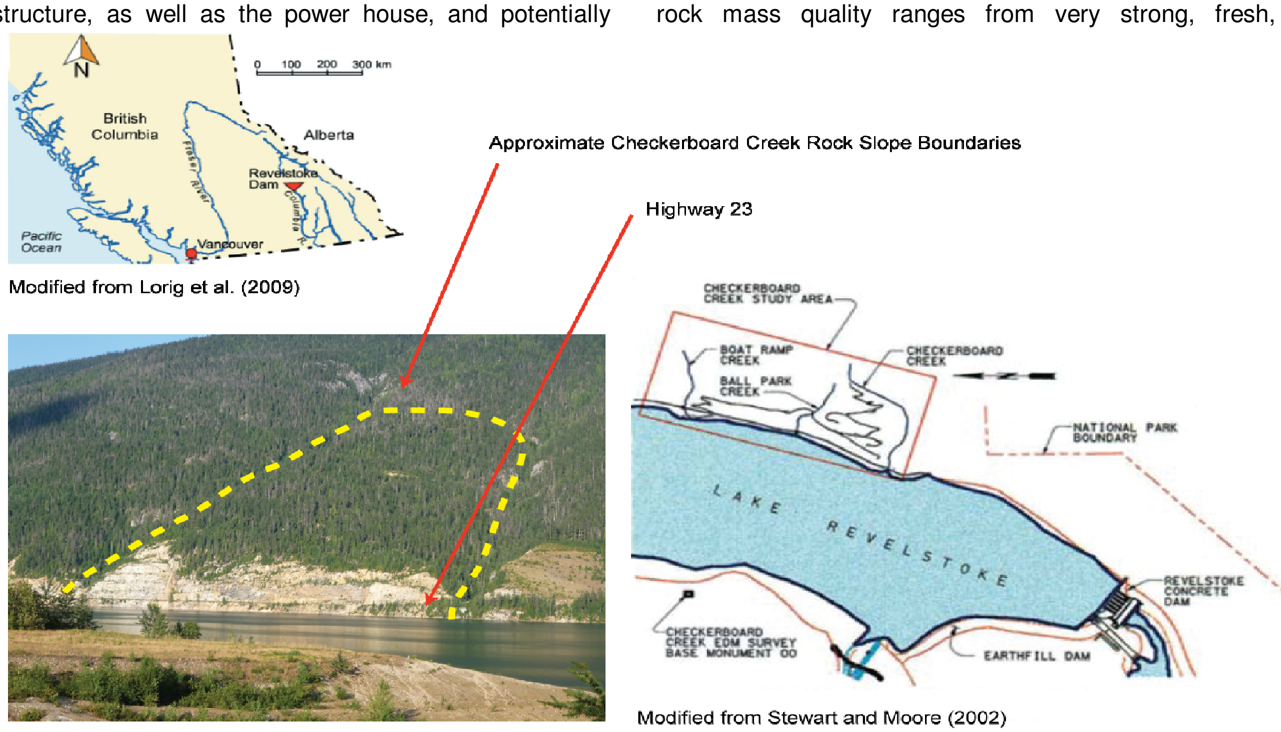


Figure 2 Location and approximate boundaries of the Checkerboard Creek rock slope.

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 3 illustrates the Checkerboard Creek rock slope geology.

The groundwater regime within the Checkerboard Creek slope is inferred from piezometric data during drilling, monitoring of multiple-piezometers (Westbay system) and observations during site inspections. These have revealed numerous, discrete, pore pressure differences of up to 40 m across short lengths which is an 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).

### 3.3 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 is constantly reviewed at Revelstoke Dam (Stewart and Moore 2002, Watson et al. 2004).

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

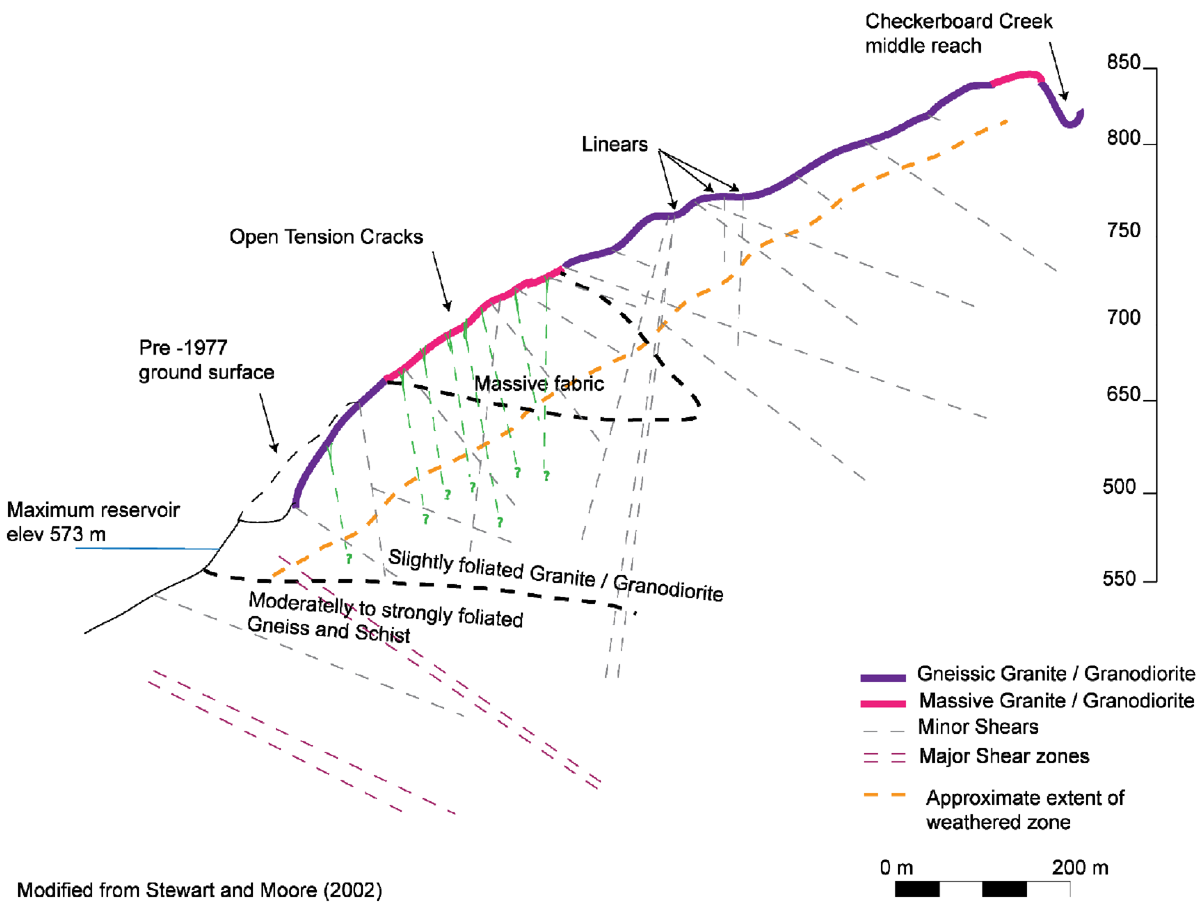


Figure 3 Simplified sketch of the Checkerboard Creek rock slope geology

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).

### 3.4 Interpretation of Deformation Mechanisms at 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 and to try to anticipate its future behaviour under extreme groundwater conditions and seismic events (Stewart and Moore 2002, Watson et al. 2004).

With the aid of these models and the information gathered from site investigations and 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 shown by the deformation measurements, particularly the rod extensometers, is more strongly correlated to seasonal temperature variations in the bedrock near the surface than to groundwater pressures. 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 lag those at the near surface bedrock by up to several months. This could indicate that the correlation between displacements and seasonal temperature changes is meaningless. However, detailed numerical analysis simulating the seasonal temperature fluctuations indicate that the induced deviator stresses produce displacements deeper than the temperature fluctuation depth and deformation patterns and magnitudes are consistent with the observations at Checkerboard Creek rock slope. 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).

### 3.5 Predictive Analysis

Watson et al. (2006) calibrated the UDEC model with the observed slope behaviour and then conducted sensitivity predictive analyses using scenarios that included rock mass strength decrease with time, increasing pore water pressures and discrete seismic events. Results from all these models 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. Even under extreme seismic events the rock mass strength was sufficient to prevent a sudden increase in displacement rates leading to slope collapse. The models also identified a zone of less than 0.5 million $m^3$ above the highway cut that was more likely to fail under seismic loading conditions (Watson et al. 2004, Watson et al. 2006). This area was identified as the slide-source for the wave studies described below.

Several wave generation studies were carried out to assess the overtopping risks for the earth fill dam. 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 travelling 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 RISK ASSESSMENT APPROACH FOR CHECKERBOARD CREEK ROCK SLOPE

### 4.1 Failure Scenarios and likelihood

Table 1 summarizes various rock slope failure scenarios based on the information obtained from site investigations, monitoring data analyses and numerical and physical 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 1 is their perceived relative 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 must be resolved by developing a formal QRA for the Checkerboard Creek rock slope. Justification of how these likelihoods were judged are also presented in the table.

4.2 Comprehensive approach

Considerable effort for understanding the mechanisms involved in the deformational process of the Checkerboard Creek rock slope has lead to our current knowledge of the danger. It has also allowed us to get a sense of the possible slope behaviour under future seismic loads and piezometric conditions.

Table 1 Checkerboard Creek rock slope failure scenarios for a time period of 100 years

| Volume Scenario | Relative likelihood | Justification |
|---|---|---|
| Rock falls (from small 1 $m^3$ to ranges of 10 to 100 $m^3$) | The most probable scenarios | Justified by the rotational nature of movements and rock mass degradation mainly on the face and above the highway cut and below the open tension cracks. Backed up by numerical models as the most sensitive area within the slope. Rocks fallen from the slope have already been observed. |
| Less than 0.5 $Mm^3$ (highway cut – rock slope toe) | Realistic scenario given slope failure occurs | 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 realistic scenario given a slope failure occurs. |
| 2 to 3 $Mm^3$ (actively deforming rock mass) | Unlikely to very unlikely | Defined by the total deforming zone interpreted from morphological evidence and instrumentation data. Includes zones where deformation rates are minimum (2 – 5 mm/y) compared to most active zones (10 – 15 mm/y) and would require sudden strength loss of the entire zone. Numerical models indicated stable conditions even under the 10 000 year return period seismic event in the area. |
| 20 – 55 $Mm^3$ (Checkerboard Creek rock slope) | Very unlikely to extremely unlikely | Ridge morphology of Checkerboard Creek rock slope considering diverse depths / % slope failed. 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. |

To carry out a risk assessment for the Checkerboard Creek rock slope, it is necessary to assess the likelihood of occurrence for each of the scenarios perceived as realistic. It is obvious now that any attempt in doing this will involve a considerable input of expert judgement, which will be very much supported by all the studies and knowledge summarized in previous sections. The nature of the slope deformation mechanisms and lack of a well developed rupture surface makes it extremely difficult to estimate the slope failure probability under specific conditions. Also, the uniqueness of the slope characteristics (i.e. geology, geometry and history of highway cut and reservoir infilling) when compared to previously failed slopes in similar contexts, also makes it difficult to correlate historical failure frequencies to the likelihood of failure of the Checkerboard Creek slope.

In order to estimate the consequences given the failure scenarios, it is necessary to understand all the elements at risk. Elements at risk will 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 (i.e. camping areas, boaters, tourists). An exhaustive analysis of the possible consequences will require knowing the location of the structures and the costs related to repairing/re-building them and the financial losses associated to the disruption of serviceability. Also, knowledge of the number of people at every location assessed will be required. This includes the populated areas, traffic through the highway, campers, boaters, etc., and how these are distributed throughout the year (temporal probability). Other aspects such as environmental losses and public perception will also have to be considered.

Table 2 shows an example of a Failure Mode and Effects Analysis (FMEA) applied to the Checkerboard Creek rock slope and considering one failure mode (rock falls). The analysis aids in the identification of not only the failure modes (in this case, the rock fall event), but also the potential consequences and elements at risk. Also, an assessment of the pre-failure signs and potential early detection is done at this stage. Note the qualitative likelihood descriptor of the failure mode is taken from Table 1. Also, a preliminary relative severity of the consequences is presented to aid the identification of the most critical failure modes when compared to their

Table 2 Example FMEA for the Checkerboard Creek rock slope – Rock fall as failure mode

| FM | Perceived Relative Likelihood | Failure Early Detection | Potential Direct Effects | Potential Indirect Effects and Wave Generation Effects | Perceived Relative Severity |
|---|---|---|---|---|---|
| Rock falls | The most probable scenario | None. Likely to be triggered by precipitation events and seismic events. | Damage / blockage of Highway 23.<br>Potential injure / life loss to highway users. | No wave generated.<br>Economic loss due to highway serviceability being interrupted. | Non severe |

relative likelihood of occurrence. These relative severity is to be further resolved by the formal QRA.

In order to carry out a comprehensive QRA, an exhaustive FMEA will be required to account for all realistic failure modes and their consequences. The FMEAs will also provide a means to detect the elements at risk and the potential for failure early detection, which will be valuable input in assessing the consequence probabilities.

The estimation of occurrence probabilities for each failure mode will require the aid of event trees. These event trees will consider the ranges in magnitude of the identified possible trigger mechanisms, and will also consider the temporal differences in slope response according to the time of the year. Estimation of consequences for the different failure modes will also require the aid of event trees. These trees will address the temporal variability of the elements at risk, pre-failure warning signs, efficiency of evacuation plans, wave generation potential and vulnerability of the elements at risk.

Finally, the likelihood of occurrence of the failure modes is combined with their estimated consequences in order to obtain a measurement of risk. Regarding the loss of life, the number of potential fatalities for each failure mode is combined with the failure mode occurrence probability to assess the potential loss of life per year. These estimation can be plotted in a F-N curve (cumulative frequency of N or more fatalities, F, against the number of fatalities, N) and compared against some acceptability criteria. As it was shown by Morgenstern (1995), there is a wide range of life risk acceptability criteria from diverse organizations and it is necessary to adopt some criteria relevant to the context where the Checkerboard Creek slope is located.

## 5 CONCLUSIONS

When the stability of a slope endangers society, engineers use some form of risk assessment to evaluate the hazard. Since the introduction of the Australian Landslide Risk Management Concepts and Guidelines in 2000, more formal risk assessment methodologies have been proposed. These methodologies have evolved into a general framework for landslide risk management that can lead to quantitative risk analyses. However, in order for quantitative risk analyses to be of value, a risk tolerance or acceptability criteria must be specified.

The Checkerboard Creek rock slope, located 1.5 km upstream the Revelstoke dam, has been widely studied and monitored since the late 70's. Extensive site investigations, ongoing monitoring of slope deformations, groundwater, temperature and precipitation, and development of numerical models; have lead to our current knowledge of the mechanisms involved in the deformation process. Moreover, our understanding of these processes allowed for the prediction of the slope behaviour under future extreme events (i.e. seismic loading and groundwater extreme elevations).

A comprehensive risk assessment of the Checkerboard Creek slope will require an exhaustive review of all the acquired information, development of FMEA, expert judgement to estimate the likelihood of failure modes and the consequences of such failure. A comprehensive quantitative risk assessment for the Checkerboard Creek slope is currently underway and the results will be reported in future publications.

## ACKNOWLEDGEMENT

This work was supported by the Canadian Railway Ground Hazard Research Program, and the Natural Sciences and Engineering Council of Canada.

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