Risk management framework and mitigation for rock fall hazard at the Clowhom River Hydroelectric Project, near Squamish, BC

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ABSTRACT
Located on the Clowhom River, approximately 27 km west-northwest of Squamish, BC, the Clowhom River Hydroelectric Project (HEP) is a set of environmentally friendly, run-of-the-river facilities that generate electricity without the creation of an upstream reservoir. The Clowhom HEP is owned and operated by Veresen.

In the initial stages of the Clowhom HEP, a rock fall hazard was identified above the location of the proposed lower intake. Subsequently, a large boulder detached from this area and tore through the trees and struck the forest service road above the intake in a location where workers had started construction. A qualitative and quantitative risk assessment of the hazard from rock fall to workers and infrastructure was carried out. The region above the lower intake and penstock crossing were determined to have an unacceptable level of rock fall risk. A model of the rock slope was developed using the Colorado Rockfall Simulation Program (CRSP). The data gained from the modelling was used in the design of a Trumer Schutzbauten rock fall fence at the base of the slope to mitigate the rock fall hazard. Detailed design of the fence, including anchor and foundation post design, were completed. The rock fall fence was installed in 2008 under the guidance of the engineering team. In 2012, this fence was significantly tested by a rock fall event that was estimated to be 12 m^3 in size. This event was completely contained by the rock fall fence. The fence even held back a boulder larger than the design size for the fence. The results were no injuries to the workers and no damage to the critical infrastructure at the intake site, but repairs were required to the fence.

RÉSUMÉ
Situé sur la rivière Clowhom, environ 27 km ouest de Squamish, Colombie-Britannique, le projet hydroélectrique de la rivière Clowhom (HEP) est un ensemble d'installations pour l'environnement amicales, fil-de-l'eau qui produisent de l'électricité sans la création d'un réservoir en amont. Le HEP Clowhom est détenu et exploité par Veresen.

Dans les premiers stades de la HEP Clowhom, un risque de chute de rock a été identifié à l'emplacement de l'apport plus faible proposée lors de l'évaluation des aléas géologiques. Par la suite, un gros rocher détaché de ce domaine et a déchiré à travers les arbres et a heurté le chemin forestier au-dessus de l'entrée et l'emplacement où les travailleurs avaient commencé construction. Une évaluation des risques qualitative et quantitative du risque d'éboulement aux travailleurs et aux infrastructures a été réalisée. La région au-dessus de la basse consommation et conduite forcée traversant étaient déterminés à avoir un niveau inacceptable de risque de chute de rock. Un modèle de la pente rocheuse a été développé en utilisant le programme de Simulation des éboulement Colorado (CRSP). Les données acquises de la modélisation a été utilisées dans la conception d'une clôture de chute de rock Trumer Schutzbauten à la base du talus pour limiter les risques de chute de rock. Conception détaillée de la barrière, y compris l'ancre et la Fondation post design, ont été achevés. La clôture de chute de rock a été installée en 2008 sous la direction de l'équipe d'ingénierie. En 2012, cette clôture a été significativement testée par un automnal de roche qui a été estimé à 12 m3 en taille. Cet événement a été intégralement par la clôture de chute de rock. La clôture même tenue en arrière un rocher supérieure à la taille de la conception à la clôture. Les résultats ont été pas de blessures aux travailleurs et pas de dommages à l'infrastructure critique sur le site d'admission.

1 INTRODUCTION

The Clowhom River Valley is located within the Coast Mountains of western British Columbia, approximately 27 km west-northwest of Squamish, BC (Figure 1). The area of interest for this study is the section flowing from Phantom Lake to Clowhom Lake that is occupied by the Clowhom Hydroelectric Project (HEP) owned by Veresen. Over this area, the valley displays the characteristic U-shape of a glacially carved valley with steep valley slopes. The valley bottom is relatively flat, widening towards Clowhom Lake, and in-filled with deposits of colluvium and alluvium. There are at least three substantial bedrock outcrops located in the valley bottom.

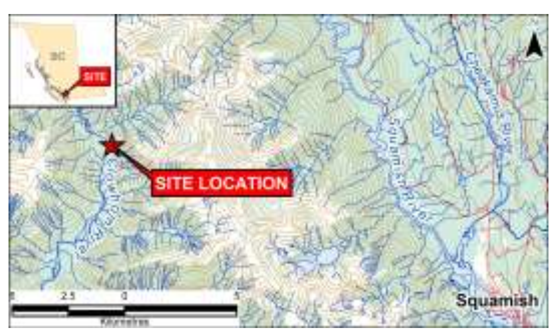

Figure 1: Site Location Map

The bedrock geology within the Clowhom River Valley consists of variably foliated hornblende quartz diorite, tonalite, and hornblende diorite intrusives, which form part of the Western Coast Plutonic Complex (Journeay et al., 2000). A thrust fault trending northwest has been mapped approximately 17 km east of Clowhom Lake.

Environment Canada rainfall records for the nearest climate station at Clowhom Falls, located 20 km southwest of the study area at an elevation of 23 m, indicate an average annual rainfall of approximately 2183 mm and average annual snowfall of about 58.7 cm. Extreme daily rainfall during the year ranges from 53 mm to 112 mm in a 24 hour period. During the winter months (December, January, February, and March), extreme snow depth ranges from 182 cm to 240 cm and extreme snowfall ranges from 23 cm to 81 cm in a 24 hour period.

Environment Canada further reports that the average daily temperature ranges from 2.1°C to 18.2°C at Clowhom Falls. During the winter months the average daily temperature ranges from 2.1°C to 5.9°C. The extreme minimum temperature recorded was -13°C in December 1983; the extreme maximum temperature recorded was 36°C recorded in August 1981.

It should be noted that the station is at Elevation (El.) 23 m which is significantly lower than the elevation of the drainage area and the main headworks that are located at El. 250 m.

2 CLOWHOM HEP DESCRIPTION

The Clowhom Hydroelectric Project (HEP) started construction in April 2008, and went into operation in late 2009. The Clowhom HEP consists of the following components (Figure 2):

- An upper hydro plant with: an intake near El. 540 m; a powerhouse near El. 350 m; and a 1955 m long penstock along the existing Forest Service Road (FSR) on the left side of the river.
- A lower hydro plant with: an intake near El. 205 m; a powerhouse near El. 110 m; and a 1250 m long penstock along the right side of the river.

There is an existing BC Hydro generating facility on Clowhom Lake, approximately 20 km (by road) south of the proposed Clowhom lower powerhouse. The transmission lines from the Clowhom development will extend to that facility.

Both hydro schemes are “run-of-river” and include:

- Headworks including a weir, sluiceway and penstock intake structure;
- Penstock through which water is transported to the powerhouse;
- Powerhouse;
- Tailrace where water is returned to the river;
- Transmission lines and switch yard; and
- Access roads.

3 SITE DESCRIPTION

The specific area of interest is the steep slope section above the Clowhom River at the location of the intake of the lower hydro plant, where a large, high rock bluff forms the eastern side of the valley. The area beneath this rock bluff is prone to impacts from rock slides and falls. This section consists of a bedrock controlled slope with a western aspect that ranges from El. 205 m to El. 1675 m.

The historic rock fall impact area above the intake is approximately 200 m wide and extends some 300 m above the FSR to the base of the rock bluffs. Within this area and below the rock bluffs, talus slopes formed from rock fall are present. The extent of rock fall can be seen from new or younger tree growth with few older or more established conifer trees. From the aerial photographs, two distinct impact zones can be seen at the base of the rock bluff. The rock at these impact zones appears to be dusted with recent shattered rock fragments and “rock powder”.

Beneath the main rock fall impact zones, several scree slopes have formed within the trees which appear to be the main pathways for rock falls. One of these paths has reached the river, depositing large, angular boulders.

High up on the rock bluffs, overhanging rock with weathered surfaces are observed and are a potential source of further rock falls. On the southern side of the historic rock fall area, a gully has formed, containing a small creek. Along this gully, a recent rock slide has occurred which has blocked the access road in the past.

4 HAZARD AND RISK ASSESSMENTS

4.1 2005 Preliminary Geohazards Assessment

An overview geohazards assessment was undertaken in 2005 for the entire Clowhom HEP, the following geomorphic process were identified (Figure 2):

- Debris flows in gullies;
- High precipitation/flooding;
- Snow avalanches;
- Rock falls; and
- Debris slides.

Figure 2: General Geohazards Map and Project Arrangement

With respect to rock fall hazards, the preliminary study concluded that of the five rock fall areas identified, the only area presenting a risk to the project infrastructure (excluding the transmission line) was located above the headworks of the lower project. As a result of this conclusion, the following were recommended:

- Conduct a field investigation during the design stage to characterize the risk of potential rock fall; and
- In examining the potential hazard to the penstock we recognized that with sufficiently thick, sloped backfill a large boulder could be deflected so as not to puncture or damage the integrity of the penstock.

4.2 2007 Detailed Rock Fall Hazard Assessment

Following the preliminary geohazards assessment in 2005, a more detailed rock fall hazard assessment was undertaken in 2007. The 2007 assessment was undertaken by helicopter and in part focussed on the inspection of high rock bluffs above the intake. In particular the assessment examined the location of scree slopes, rock flour on the bluffs and vegetation changes below rock bluffs (in that mature conifers were absent). The results of the 2007 rock fall hazard assessment indicated that further large rock slides and rock falls, which would be capable of reaching the Clowhom River, would be considered likely in the 50-year design life of the Clowhom HEP. This assessment recommended that the intake structure, upstream portion of the penstock and the penstock river crossing be protected to mitigate against the risk from rock fall.

As the penstock was to be buried from the intake until the point at which it crosses the Clowhom River (about 100 m downstream from the intake), it was recommended to align the location of the penstock crossing with a stand of mature, coniferous trees that are growing within the rock fall debris area. This stand of trees represents an area on the slope which is less prone to impact from rock fall. It was further concluded that these trees would also provide limited protection should further rock fall occur.

This detailed assessment resulted in the following recommendations that are shown on Figure 3:

- At the location of the penstock crossing, a 20 m long rock fall barrier (without anchors) be constructed to intercept rock fall and prevent strikes to the penstock;
- At the location of the intake weir, a 30 m long rock fall fence be installed adjacent to and upslope of the FSR. It has been proposed that the intake and the FSR will be separated by a 45° slope created using rock fill. At the base of this slope, a second catch fence should be installed as a final line of rock fall protection. The second fence will be 20 m long and constructed with up-slope anchors;
- As per the above bullet, it was noted that there was a 10 m wide access area between the base of the rock fill slope and the intake structure. Therefore, a second fence should be installed to keep this area open for unrestricted access. As an alternative to the 20 m long second fence, a 3 m high, 3 m thick lock-block wall could be created and filled with rock fill. This would provide substantial rock fall protection, but would intrude upon the access space above the intake; and
- It was also recommended that the control room at the intake be partially buried to protect it from rock fall. On the slope side of the building, the roof should be reinforced and angled into the ground so that any boulders roll over the building. The roof should therefore be a chevron shape in three dimensions so that boulders naturally spill off the sides.

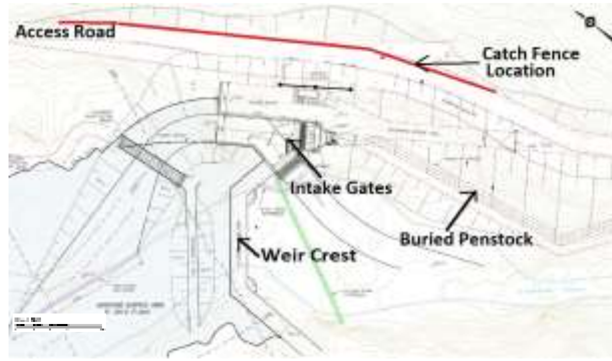

Figure 3: Plan of Intake Showing Catch Fence Locations

5 2008 ROCK FALL EVENT

Before the measures outlined in the previous section could be constructed, a rock fall event occurred on May 5, 2008, initiating just to the south (downstream) of the proposed location for the lower intake weir. The rock fall event deposited several small boulders on the FSR, and one boulder impacted the outer edge of the FSR with sufficient force to create a small crater (Figure 4). Figure 5 shows a general overview of the rock bluff source area.

Figure 4: Impact Location of Rock Fall

The source of the recent rock fall appears to be from a wedge style failure. It is apparent from the character of the visible open joints within the rock on the southern side of the wedge, that further significant rock fall could occur from this location.

In terms of potential causes, the climatic conditions at the site indicate that any of: heavy rainfall, freeze-thaw, wind, snowmelt, or channelled run-off could induce rock fall, in connection with the observed adverse joint pattern and high degree of weathering, as suggested by studies carried out by McCauley et al. (1985).

The occurrence of this event prompted the completion of a qualitative and quantitative risk assessment, and a risk control plan that are described in the sections below.

Figure 5: Overview of the Intake Location and Rock Fall Area

6 FOLLOW UP RISK ASSESSMENT AND RISK CONTROL PLAN

6.1 Qualitative Risk Assessment

Based on the results of the detailed rock fall hazard assessment and the 2008 rock fall event, the likelihood of a rock fall event impacting areas of the FSR and the adjacent proposed penstock alignment, has been categorised into Low, Medium and High. The probability has been assessed for the time period during the 2 years of construction. In the long term almost all of the area below the rock slope could be impacted by rock fall. The probability is presented as green (Low), yellow (Medium) and red (High), respectively on Figure 6.

The probability of a rock fall event is highest below the main talus slope and within the gully to the south (debris flow and snow avalanche hazards are present, as well as rock fall). Another high risk location occurs where the smaller talus slope at the base of the main rock bluff has created a chute for larger boulders to travel down to the FSR. Other locations have generally been shown as medium and low probability.

In order to attempt to qualitatively assess the risk both to workers and to infrastructure on the site, we have considered three time intervals: 1) the short term of 2 to 3 months; 2) the medium term which is considered to continue until the completion of construction of the works (2 years); and 3) the long-term which continues until the end of the service life of the works and plant (50 years).

For each of the three time intervals the risk from a rock fall event will be different. In the initial 2 to 3 months of construction (i.e. in the short term), work activities would be at the intake site and penstock alignment, During this period, work would occur directly below the location of the recent rock fall, as this is close to the proposed location of the penstock crossing. In the short term therefore, potential consequences of an event are; loss of life, injury or damage to vehicles and equipment on the FSR or below the FSR during penstock construction.

Figure 6: Subjective Probability of Rock Fall Over the 2 Year Construction Period. Low=Green, Medium=Yellow, High=Red

In the medium term the penstock crossing and intake structure will be completed and the FSR will be in constant use. Potential consequences of an event are; loss of life, injury or damage to vehicles and equipment on the FSR or below the FSR during construction.

In the long term, the penstock will be buried. However there are three key structures which could be damaged by a rock fall event in the long term: the penstock at the crossing; the intake weir and associated structures; and the intake control room.

Risk is defined as the expected consequence of an event multiplied by the probability of that event occurring. Our qualitative risk assessment is based on a simple matrix defined by a range of low to high consequences and probabilities of rock fall impacting the FSR, as shown in the Table 1.

Based on Table 1, risk ratings have been calculated for different work elements at different stages, and are summarized in Table 2.

In the assessment of risk summarised in Table 2, long term damage to structures has been defined as a lower consequence than potential loss of life. This definition is dependent upon the owner’s tolerance to risk, as the consequence of damage to the intake or penstock may be considered to be high from the owner’s perspective.

The result of the risk assessment shows that the penstock crossing, both in the short and medium terms, is at very high risk. The penstock near to the crossing has a very high risk during construction, but in the long term it will be buried, and has a medium risk. The FSR has a very high risk both in the short and medium terms. In the long term, the intake structure and the penstock crossing have a high risk.

RISK RATING		Consequence		
		Low (1)	Medium (2)	High (3)
Probability	Low (1)	Very Low (1) (acceptable)	Low (2) (tolerable)	Medium (3) (tolerable)
	Medium (2)	Low (2) (tolerable)	Medium (4) (tolerable)	High (6) (intolerable)
	High (3)	Medium (3) (tolerable)	High (6) (intolerable)	Very High (9) (intolerable)

Table 1: Risk Matrix (modified from Porter and Morgenstern, 2013)

Structure	Time Interval		
	Short Term (initial 2 to 3 months of construction)	Medium Term (during 2 years of construction)	Long Term (post construction)
Penstock Crossing	(3 x 3) = 9 (VH)	(3 x 3) = 9 (VH)	(2 x 3) = 6 (H)
Penstock near to crossing	(3 x 3) = 9 (VH)	(3 x 3) = 9 (VH)	(1 x 3) = 3 (M)
Penstock near to Intake	(3 x 2) = 6 (H)	(3 x 2) = 6 (H)	(1 x 3) = 3 (M)
Intake Structure	(3 x 1) = 3 (M)	(3 x 2) = 6 (H)	(2 x 3) = 6 (H)
Forest Service Road	(3 x 3) = 9 (VH)	(3 x 3) = 9 (VH)	(1 x 3) = 3 (M)

Table 2: Calculated Risk Ratings for the Elements at Risk (Notes: First multiplier is consequence; second is probability; VH=Very High; H=High; M=Moderate).

The above qualitative assessment results in high and very high risk at most of the structures of interest. Because the estimated risk is considered intolerable, a detailed quantitative risk assessment is warranted.

6.2 Quantitative Risk Assessment

The quantitative risk assessment from rock fall is focussed on the FSR, as this road will be in constant use during construction, with results related to relevant acceptability criteria.

Two measures of risk are considered: risks to individuals and risks to groups (or societal risk). Individual risk addresses the safety of individuals who are most at risk in an existing or proposed development (Porter and Morgenstern, 2013). Societal risk addresses the potential societal losses as a whole caused by total potential losses of people in the community from a hazard event. When considering the exposure to a single rock fall event, risk is calculated according to Equation 1.

$$R = P_H \times P_{S:H} \times P_{T:S} \times V \times E \qquad (1)$$

where:

R = risk
P_H = annual probability of the hazard (i.e. rock fall event) occurring
$P_{S:H}$ = spatial probability that the rock fall event will reach the individual
$P_{T:S}$ = temporal probability that the individual will be present when the rock fall event occurs
V = the vulnerability, or probability of loss of life if an individual is impacted
E = the number of people at risk; equal to 1 for individual risk

Described in the subsections below are the results of our risk assessment for the case of rock fall onto a moving vehicle, as that is considered our most likely scenario. We did not examine the case of a stationary vehicle hit by rock fall (as “No Stopping” signs were posted), or the case of a vehicle striking a fallen rock on the FSR.

6.2.1 Frequency Analysis

The frequency analysis is based on the following assumptions:

- As a worst case assessment given uncertainty, one rock fall event occurs per week and reaches the FSR with sufficient force to be able to cause loss of life;
- The danger zone equates to the two high probability areas shown on Figure 6, plus the in-between moderate probability area.
- One vehicle passes through the danger zone every 5 minutes (in either direction) throughout a 12 hour period.
- The FSR is 6 m wide, and is considered to be one-lane.

This implies there are a total of 52 rock falls per annum or 0.14/day, which equates to an average frequency of rock falls (NR) onto the single lane FSR of 0.14/day.

6.2.2 Consequence Analysis -Temporal Spatial Probability

The probability of a vehicle occupying the length of road impacted by the rock fall is given by Bunce et al. (1997) and Fell et al. (2005):

$P_{T:S} = N_V/24 \times L/1000 \times 1/V_V$ (2)

where:

$P_{T:S}$ = temporal probability that the individual will be present when the rock fall event occurs

N_V = average number of vehicles/day

L = average length of the crew cab of the vehicle in metres

V_V = velocity of the vehicle in km/hr

For this analysis, the following assumptions are applied:

- The typical vehicle using the FSR is a crew-cab pick-up truck with an average cab length of 4 m.
- The average velocity of the vehicle over this section of the FSR is 30 km/hr.

Based on 144 vehicles passing through the danger zone each day, Equation (2) implies that $P_{T:S}$ is 8 x 10^{-4}.

6.2.3 Consequence Analysis - Vulnerability

For the determination of vulnerability of persons in the vehicles (V), we have used a value of 0.3 based judgement and Bunce et al. (1997). With two people on average occupying the truck travelling through the danger zone on the FSR, this means there would be 0.6 persons (say 0 to 1) killed if the truck was impacted by a rock fall event (i.e. 0.3 x 2 people in the cab).

6.2.4 Risk Estimation

From Fell et al (2005), the annual probability of the person most at risk in losing his/her life by driving along the FSR is:

$R = P_{LOL} = P_S \times V$ (3)

where P_{LOL} = the annual probability that the person will be killed.

P_S is the probability that one or more vehicles is impacted by rock fall, as defined by Bunce et al. (1997) and expressed as:

$P_S = 1 - (1 - P_{T:S})^{NR}$ (4)

Equation 3 implies that for the individual most at risk to lose their life from a rock fall event impacting the FSR, (P_{LOL}) is 3.4 x 10^{-5}.

The annual probability of an accident to a particular vehicle is 9.3 x 10^{-8} (3.4 x 10^{-5}/365).

6.2.5 Risk Evaluation

Leroi et al. (2005) summarize the various international individual tolerable risk criteria. Commonly, the tolerable risk for existing slopes is 10^{-4}/annum for the member of the public most at risk (e.g. the Australian Geomechanics Society, 2000). One exception noted in Leroi et al. (2005) is the criteria established by the Roads and Traffic Authority for NSW Australia that specifies 10^{-3}/annum. Using either of these values indicates that the risk from rock fall over the danger zone along the FSR is acceptable to the individual.

Societal risk estimates are typically presented on graphs showing the expected frequency of occurrence and cumulative number of fatalities, referred to as F/N plot (Figure 7). The accepted societal risk for one life lost is 10^{-3} / annum (see Figure 7). The graph is subdivided into four areas: unacceptable risk; tolerable risk that should be reduced further if practicable according to the as low as reasonably practicable (ALARP) principle; broadly acceptable risk; and a region of very low probability but with the potential for >1000 fatalities that require intense scrutiny. Porter and Morgenstern (2013) indicate that from the perspective of potential loss of life from a landslide (or in this case, rock fall), development is typically approved if it can be demonstrated that the rock fall risk falls in the ALARP or broadly acceptable regions on an F/N plot.

The estimated probability of one or more lives lost is 5 x 10^{-3}/annum (144 x 365 x 9.3 x 10^{-8}), which plots in the Broadly Unacceptable region of the F/N plot (Figure 7) due to rock fall within the study area.

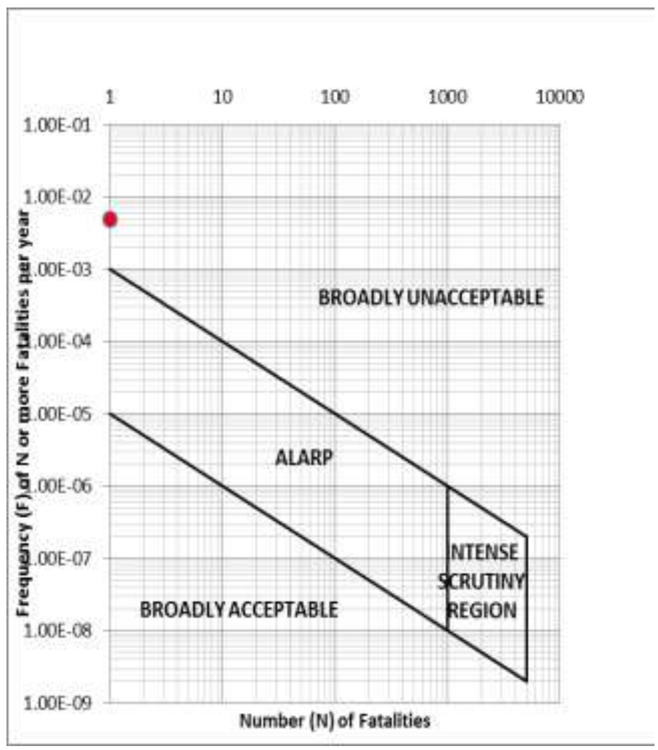

Figure 7: F/N plot considering road users along the FSR (criteria modified from Hong Kong Government Planning Department (1998).

Given this result, and the fact that workers constructing the penstock on the downslope side of the FSR will have greatly increased exposure time to the rock fall hazard that this work would necessitate, the workers could be at an intolerable risk in carrying out this work. Similarly in the long term, critical infrastructure such as the penstock crossing and the intake is at an intolerable risk. Consequently, risk control measures should be constructed and maintained in order to reduce the risk to a tolerable level.

6.3 Risk Control

The hierarchy of risk control methods ideally are: 1) elimination, 2) substitution, 3) engineering controls, 4) administrative controls, and 5) PPE (personal protective equipment). In the case of risk control from the rock fall hazard at Clowhom, it is clearly not practical to eliminate the risk (i.e. move the intake / penstock). Therefore engineering control measures have been considered as well as administrative control measures (safety practices / procedures).

There are several risk control measures which could be utilized to protect the workers and critical infrastructure both during the construction and in the long term operation of the HEP. Options considered included:

- Trim Blasting of the Source Area – using a combination of helicopter and roped access. Trim blasting has the advantage of removing the current hazard, at the location of active rock fall. The disadvantage is that there are numerous locations of historic rock fall that are not currently active, but could potentially be a hazard during the course of the works or in the long term.
- Safety Controls – procedures and criteria have been developed that restrict work activities and access around the FSR and below the rock slope as much as is practicable. They also indicate where foot access is prohibited altogether during intense rain fall events, and at times of the year / day where freeze thaw is possible, such that work and access is restricted.
- Physical Barrier - for the short and medium term, a physical barrier could be placed along the FSR. In the long term a substantial physical barrier could be constructed at the penstock crossing or at the intake weir. However, given the historical size of rock fall this barrier would have to be very large, and cannot practically be constructed to protect workers during construction.
- Trumer Catch Fence - A Trumer catch fence could be installed along the road in order to protect the workers during construction, and to protect the critical infrastructure in the long term. One of the disadvantages of the Trumer fence is that every time there is a rock strike, the fence would require maintenance.
- Diversion Bund - the concept of a diversion bund is not to stop the rock fall but to alter the course or travel path of the rock so that it misses the structure to be protected. For example above the penstock crossing a chevron shaped, engineered fill bund could be constructed so that a rock which would otherwise strike the penstock, would be deflected off the bund and miss the penstock. The advantage of such bunds is that they are a long term solution to rock fall, they would also require very little maintenance (unless several rock falls had deflected off them). The disadvantage is that in the short and medium term they offer no protection for the workers or equipment.
- Cover Structures - the intake could be covered with a reinforced slab (with possible earth fill and hydro-seeding for further protection and aesthetics) such that the concrete intake structure would be protected. However, this is a long term solution and does not mitigate against the short and medium term risk to workers.

There is not one control measure that will reduce the risk from rock fall to an acceptable level in the short, medium and long term. It is therefore considered that several of the control measures presented should be utilised in order to protect the workers during construction, and protect the infrastructure in the long term.

7 ROCK FALL MITIGATION

7.1 Design

The optimal risk control measures were determined to be the Trumer Catch Fence as based upon the 2007 and 2008 assessments and analyses, in connection with Safety Controls. The design is shown in Figure 8.

The design is based on the size of historic rock fall blocks observed in the Clowhom River (0.5 to 1.5 m in diameter) and rock fall modeling using the Colorado Rockfall Simulation Program (CRSP). Based on these results, it was concluded that a 90 meter long, 4 meter high, 1000 KJ rock fence should be installed. The design included anchors and guy cables on the upslope side of the fence using the upslope anchors fence design (Fence model #TS-1000-ZD).

Although a post fence option could have been constructed (with no upslope anchors), the substantial overturning forces potentially induced into the post foundations meant that this foundation would have been prohibitively expensive. Therefore an upslope anchor system on the upslope side of the FSR was used.

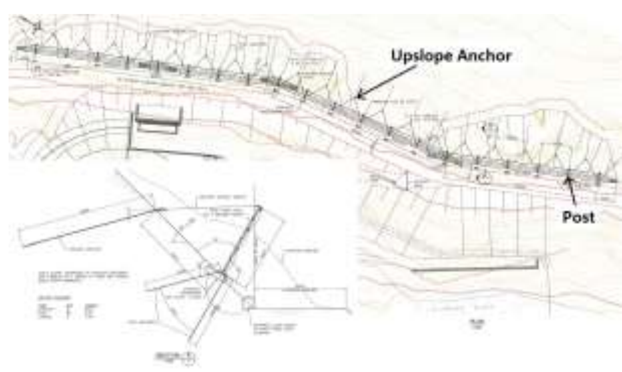

Figure 8: Rock Fall Fence Plan and Section

Design issues encountered were associated with the installation of the fence anchors as they needed to be drilled, installed and grouted into loose talus. Two issues arise when drilling into a slope of this material: first, loose material in the talus slope collapses into the anchor drill hole, thus making it difficult to pull the drill bit and stem out of the hole prior to anchor installation; secondly the loose talus material exhibits poor rock to grout bond strength. Significant volumes of cement grout could be lost into the voids of the talus material; thus, it is very difficult to determine the degree to which the anchors have been properly encapsulated by grout.

Both of these problems were resolved with the use of self-grouting, Dywidag hollow stem anchors with a sacrificial drill bit. Pre drilling of anchor holes is no longer necessary with a sacrificial drill bit and grout can be pumped continuously through the hollow anchor and bit. These anchors are suitable for use in coarse talus soils and provide for a more reliable, efficient means of anchor installation and grout encapsulation in difficult ground conditions

7.2 Construction

The rock fall fence installation design was carried out to the standards specified and inspected by Trumer Schutzbauten Canada Ltd. (Trumer). Trumer provided the main components of the rock fall protection system, including the interception structure (primary net and additional layer), support structure (posts, ground plates and guidance of ropes), connecting components (longitudinal bearing ropes, longitudinal supporting ropes, and retaining cables) and the energy dissipating devices.

The first step of the fence installation was to excavate the slope above the FSR to a temporary angle of 1:1 (horizontal to vertical), allowing accessibility for upslope anchor installation. Excavation of the slope also allowed for the removal of unstable boulders directly above the fence position.

Figure 9: Drilling and Grouting for Fence Anchors

The rock fall fence spans a length of 90 meters across the risk zone and consists of 77 anchors in total. The anchors can be differentiated into three separate sets, comprising of upslope anchors, post anchors and lateral anchors. The upslope anchors are installed at 15˚ to the horizontal in an upslope direction. Posts have two anchors, one installed at 15˚ to the horizontal in an upslope direction, the other installed perpendicular to the slope (at 60˚) in an upslope direction. Lateral anchors are drilled at 45˚ from horizontal angled at 90˚ from the slope. The anchor support system was constructed using 3 meter long Dywidag R38N hollow bars.

The fence design consists of 20 post anchors located a minimum of 3.5 meters horizontally upslope from the existing FSR and spaced 10 meters apart. Figure 9 shows the installation of an upslope anchor. There are 21 upslope anchors in total, located five to six meters upslope from the center mark between the post anchors. Figure 10 shows upslope and post anchors after installation. Eight sets, 16 anchors in total, of lateral anchors were installed to a depth of 7.5 meters each.

Figure 10: Installing the Fence Posts

The anchors were installed using an Air Track Drill Rig with an extending boom. Post anchors and lateral anchors were easily reached by the drill rig from the existing ditch along the edge of the FSR. At the locations where the drill rig was unable to climb the talus slope to reach upslope anchors for installation, benches were constructed above the ditch using logs and fill material derived on site. These benches allowed the drill rig to safely climb higher up the slope to reach the upslope anchor positions. In order to comply with the other works happening on the project site, including traffic passing along the FSR, the drill rig and benches were positioned in a fashion to prevent obstruction to other vehicles.

Drilling operations and anchor installation were monitored daily to assess the alignment of the drill rig in terms of inclination and orientation. In order to ensure stable and secure anchors, the final drilling depth varied. When solid rock was encountered at the end of the drill hole, the anchor length was reduced as a good bond could be formed with the anchor and rock.

Due to the issues encountered when drilling into talus material, only the down slope post anchors were pre-drilled. These anchors were pre-drilled to a depth of 3 meters. Post anchors were drilled at a much steeper angle than the other anchors, making it much easier to pull the drill bit back out of the talus material. Drilling at lower angles in the loose material made it harder to pull the drill bit back out due to collapsing of the hole as drilling progresses deeper; accordingly, the rest of the anchors were not pre-drilled.

Once the anchors were installed to their design depths, a 1:1 mix of Mircosil® Anchor Grout to water was injected into the drill hole through the hollow-stem anchors. During grouting, the anchors were pulled out and pushed back into the drill hole ensuring a thorough distribution of grout in the hole.

Following the installation of the lower post anchors, micro piles were installed. The micro piles were 1.5 meter galvanized steel tubes with several perforations throughout. Micro piles were installed by sliding them overtop of the anchors and down to the level of the surface material. Once set in place, grout was poured into the pile until it reached the top of the pile. The perforations in the piles allow the grout to travel into the talus to ensure a good distribution of grout and strong bond to anchor bond near surface. Pull-tests were performed throughout the anchor installation to ensure that anchors met their design loads. Of the 77 anchors installed, 15% were tested to 110% of their design loads. The design loads for post anchors, lateral anchors and upslope anchors are 110 kN, 170 kN and 202 kN respectively. Anchors were chosen at random for testing.

Once anchor installation was complete, footings for the post anchors were constructed. Since post footings require mass concrete, construction was relatively straightforward. Shallow excavations made around the already installed post anchors and plywood formwork was put in place in the wet concrete. Reinforcing steel was installed in the footings and concrete was then poured to the top of the framework.

8 THE 2011 TEST

A rock fall event occurred in December 2011 that tested the Trumer Catch Fence. Although it was not possible to determine the velocity of the boulders, the photographs in Figures 11 and 12 suggest that the event may have been beyond what the fence was rated for; however, it seems to have performed extremely well given there were no impacts to the FSR or the headworks. The largest boulder in the fence was about 2 m in diameter and the rock fall reportedly initiated at least 300 m upslope of the fence.

Figure 11: Rock Fence Damage as Viewed from Above

Figure 12: Rock Fence Damage at Road Level

9 CONCLUSIONS

The risk management framework presented in this paper was used successfully in the risk assessment and management of the rock fall hazard at the Clowhom HEP's lower intake.

The findings of the rock fall hazard analysis and characterization showed that the large steep rock bluff on the eastern side of the valley above the lower intake site was a potential source location for rock fall events throughout the lifetime of the project. The analysis of the rock fall event that occurred in the spring of 2008 further justified that the risk of a boulder travelling with sufficient force to cause injury or death to a person in its path is high.

The qualitative risk analysis indicated that the penstock crossing, the penstock near the crossing, the penstock near the intake, the intake structure and the FSR were at intolerable levels of risk to future rock fall events throughout the construction period. It was also determined that the penstock crossing and the intake structures were at intolerable levels of risk throughout the design life of the project. Further to these findings, the quantitative analysis provided a more detailed risk assessment to the workers travelling along the FSR throughout the lifetime of the project and to the workers on the FSR throughout construction of the penstock alignment. It was again determined that an intolerable risk existed to the workers working below the FSR and therefore mitigation measure needed to be put in place to lower the risk from rock fall.

An overall ratings system used to classify different mitigation measures from most effective (in terms of period of time over which protection is provided, cost, and maintenance) to least effective proved to be a reliable way to determine the most suitable mitigation option. This comparison lead to the design and installation of a 90 m long, 4 m high, Trumer rock fall catchment fence located above the FSR over the danger zone.

Construction of the lower penstock alignment continued according to schedule following final construction of the rock fall fence. We are aware that a damaging rock fall occurred in late 2011 that was contained by the Trumer catchment fence.

For the conservative quantitative risk estimation in this paper, the total annual number of rock falls was estimated to be 52 (one rock fall per week, annually). For a more definitive assessment of the return period of large scale rock fall that would cause damage to the infrastructure at the lower intake site, further field investigation would be required. Such investigations could include: examination of the tree growth along the extent of the historic rock fall area, and assess the frequency that boulders in the river are removed by flooding events.

Further studies and classification of the annual large scale rock falls would provide a better understanding of the hazard and risk to the intake structures, thus leading to further consideration to remedial work that could be undertaken to protect the works in the long term.

ACKNOWLEDGEMENTS

The writers would like to acknowledge the contribution of a number of individuals to the paper, including the owner's representative Mr. Robert Kulka P.Eng., Operations Manager at Veresen, Mr. Cory Williams P.Eng., Senior Civil Engineer at Canadian Projects Limited, and Mr. Ahren Bichler of Trumer Schutzbauten. Construction review was undertaken by Mr. Jeremy Groves EiT.

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