PÉRIBONKA SPILLWAY ROCK CUT – ROCK FALL SIMULATIONS AND HAZARD PROTECTION MEASURES

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RÉSUMÉ
L'aménagement Péribonka est situé au nord du Lac St-Jean, province de Québec, Canada. Cet aménagement comprend, entre autres, un évacuateur de crues dont une partie de l'excavation atteint une hauteur finale de 80 m et une longueur de 400 m. À cet endroit, le massif rocheux est constitué d'anorthosite avec quatre familles de joints largement espacés. Avec les dynamitages, la présence de ces joints a contribué à la perte de roc en bordure des banquettes. Peu avant la période de dégel, certaines craintes existaient quant à la possibilité qu'une chute de blocs, issue de la partie supérieure du talus, se produise et affecte la sécurité des travailleurs. De plus, la sécurité à long terme était un aspect important puisqu'en plus de la présence de la structure de béton de l'évacuateur, la route permanente d'accès au barrage et à la centrale passe au pied de ce talus. Cet article présente les différentes étapes suivies pour la sélection des mesures de protection et certaines recommandations sont proposées pour la conception de futures excavations.

ABSTRACT
The Péribonka Hydroelectric Project is located North of Lac St-Jean in the Province of Québec, Canada. The power plant includes, among others, a spillway rock cut section with a height of approximately 80 m and a length of 400 m. The rock mass joint system contributed to a loss of rock at the benches because of the blast induced damage along the edges. In relation to the coming spring-thaw, concern existed regarding the possible hazard to the workers from rock falls originating from the rock cut located directly above the spillway construction zone and the permanent access road to the dam and the powerhouse. This paper addresses the different stages in the selection of the protection measures with some recommendations for the design of similar future channel excavations.

1. INTRODUCTION

The Péribonka Hydroelectric Project is located North of Lac St-Jean in the Province of Québec, Canada (Figure 1). The power plant includes, among others, a main rock fill dam 82 m high and 773 m in length, a reservoir of 32 km^2, an underground three unit power house of 385 MW generating capacity and a spillway with 5300 m^3/s discharge capacity (Figure 2). The powerhouse is presently in operation.

The spillway rock cut section on the right bank of the Péribonka River has a height of approximately 80 m and a length of 400 m. The spillway cut is designed with six or seven benches (depending on the location) at cut faces angle of 76°; the height of each face is 12 m and the berms are 6 m wide. The rock at the spillway location is an anorthosite with three widely spaced joint sets. The presence of these joints, together with blast induced damage to the rock, resulted in loss of rock along the crests of the benches. This condition led to the fact that, after completion of the excavation, the spillway rock cut did not represent the intended design profile; particularly, the width of the benches was far less than required in the design. The completion of the spillway excavation coincided with the beginning of the winter 2005 and the construction of the spillway concrete structure was planned to start early in the following spring. In relation to the spring-thaw, concern existed regarding the possible hazard to the workers from rock falls originating from the rock cut located above the construction zone.

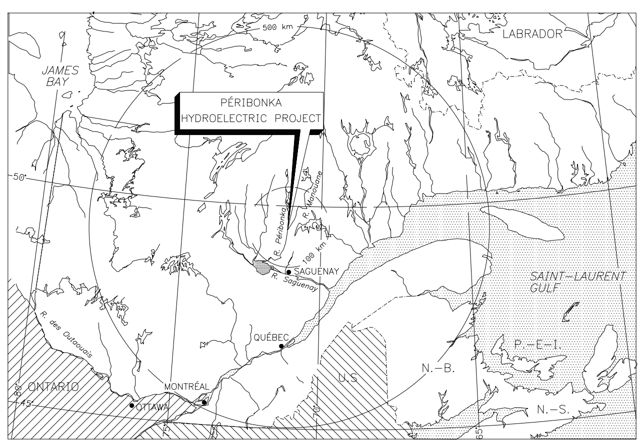

Figure 1. Location of Péribonka Hydroelectric Project

In : J. Locat, D. Perret, D. Turmel, D. Demers et S. Leroueil, (2008).
Comptes rendus de la 4e Conférence canadienne sur les géorisques: des causes à la gestion.
Proceedings of the 4th Canadian Conference on Geohazards : From Causes to Management. Presse de l'Université Laval, Québec, 594 p.

It was deemed necessary to implement rock fall control measures prior to the start of the spillway concreting to insure safety and avoid any delay in the construction. Also, long term safety is important as the spillway concrete structure, the permanent access road to the dam and the powerhouse are located along the toe of the slope.

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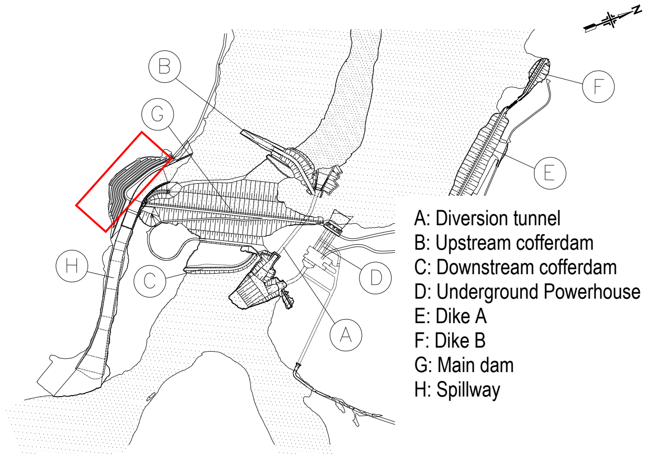


Figure 2. General layout. Spillway rock cut shown in box.

2. SITE DESCRIPTION

2.1 Geology

The Lac-St-Jean region is located in the Grenville geological province. Regional geology at the site is dominated by the Lac-St-Jean anorthosite series which is mainly composed of gabbros, norite, gabbronorite and troctolite (Laurin & Sharma, 1975). On the tectonic aspect, many brittle faults are present in the region and they are related to the formation of the Saguenay graben.

The rock in which the cut has been made is a fresh, very strong, blocky, grey anorthosite; there is no significant faulting. The excavated faces have been mapped in detail and stereonets of the joint orientations have been prepared. Prior to the construction, an investigation program was carried out and an opto-acoustic geocamera was used. From the camera borehole surveys, four major joint sets were identified (Table 1). From the site mapping, the plotted stereonets show that three of the joint sets form wedges that have lines of intersection that dip out of the face. As excavation progressed, rockbolts were installed as required to ensure the stability of the identified wedges.

Regarding the groundwater table, the ice formation on the faces may indicate that the water table was intersected by the excavation. Plotting of the ice locations on the faces provides information on the possible position of the water table, and the joint sets along which the ground water flows.

Table 1. Joint Set Orientations

Joint Set No.	Dip Direction	Dip
1	133	17
2	295	32
3	030	42
4	088	77

3. SPILLWAY DESIGN

The spillway is oriented NW-SE and is excavated in the steep hillside on the right bank of the Péribonka River. It should be mentioned that the spillway excavation section located above the reservoir level was planned to serve as a quarry to provide rock material for the construction of the main dam. The location of this large excavation is shown in Figures 2 and 3.

Figure 3. Péribonka spillway excavation. Spillway concrete structure zone in box.

3.1 Bench excavation parameters

The original study parameters for the design of spillway excavation are presented in Table 2. The bench height is the vertical distance between successive horizontal berms. The width is the horizontal extension of the berm, and the face angle is measured from the horizontal.

Table 2. Bench excavation study parameters and related excavation quantities

Option	Bench Parameters			Excavation volume	
	Height (m)	Width (m)	Angle (deg)	Rock (m^3)	Overburden (m^3)
Initial	10	6	90	295 000	58 000
1	10	6	70	440 000	77 000
2	12	10	70	1 300 000	178 000
3	10	6	60	872 000	119 000
4	10	6-7	70	630 000	96 000
Selected	12	6	76	n/a	n/a

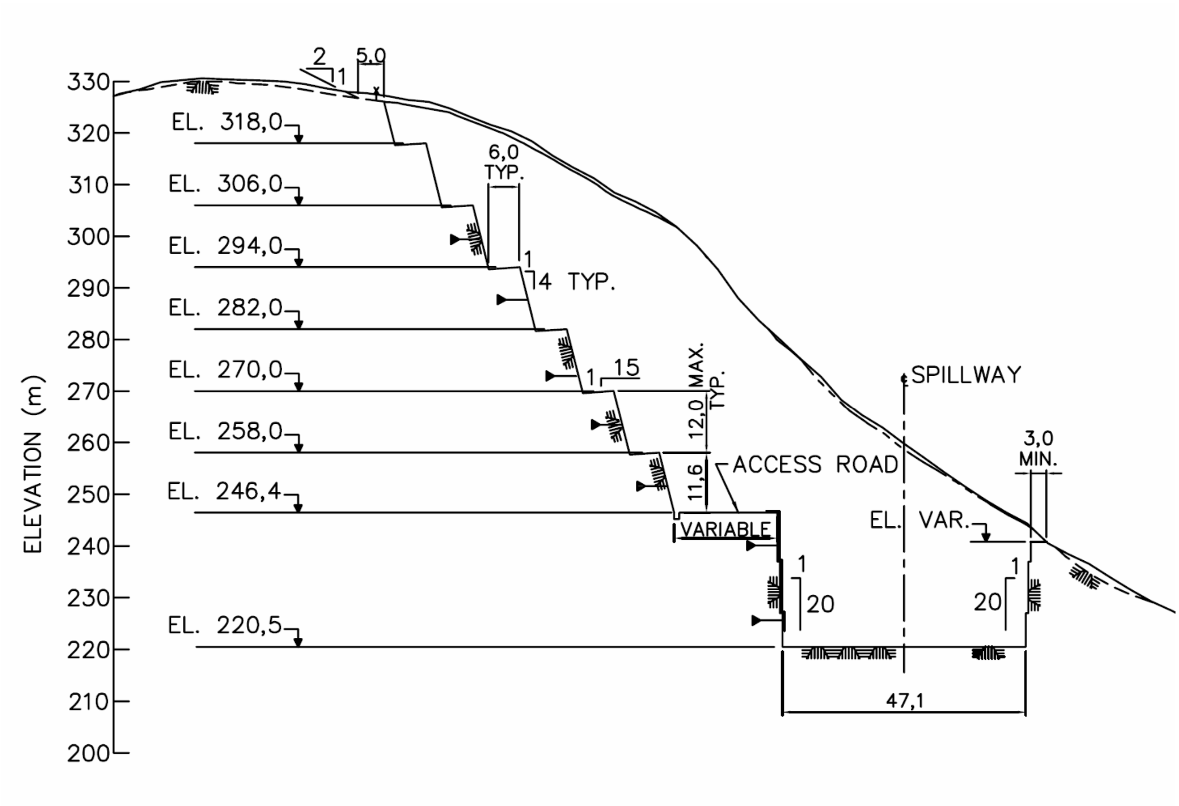


Figure 4. Typical cross-section of the spillway rock cut.

For the initial option and Option 1, an analysis was performed of possible wedge formation in the excavated faces. The analysis showed that, in many cases, the intersection of the different joint systems with the excavation face of the initial option produced unstable wedges. For the other options with the shallower face angles, no kinematically feasible wedges were formed.

The typical cross-section of the selected option is shown in Figure 4. The volume of rock obtained with this configuration is in the order of 1 000 000 cubic meters. Figure 5 shows a 3-D representation of the benches with an indication of the location of sections at Chainages 230 and 260; simulation and in situ testing were conducted at these sections.

3.2 Resulting excavation profile

Excavation works at the spillway proceeded using the pre-shearing blasting parameters presented in Table 3. It should be noted that for pre-shearing, the explosive load per delay is significantly less than the maximum value of 150 kg per delay generally required in the specifications. Also, the maximum peak-particle-velocity (PPV) accepted is 150 mm/s at 30 m distance from the blast unless there are concrete structures to be built or grouting to be conducted; for such cases, the PPV is limited to 50 mm/s.

lost.

Table 3. Pre-shearing blasting parameters at the spillway

Elevation	Blast hole Diameter (mm)	Blast hole Spacing (mm c/c)	Bench Height (m)	Explosives Mass detonated (Max. Kg/delay)
Below 246,4	60	600	10	150
Above 246,4	90	900	12	150

In the complete excavation sequence, rock blasting was followed by mechanical and manual scaling. In the spillway cut, above el. 246,4, the cleaning of the excavated faces was to be performed if requested to facilitate the inspection of the slope faces.

Upon completion of the excavation, the final profile of the excavation was not as designed. Figures 6a and 6b show the resulting section of the slope face, at Chainage 260 and 230 respectively, from el. 245 to el. 320 approx. It can be seen that the 6 m wide berms are almost inexistent at many benches, or that the sharp crests were

If a block detaches from the top of the slope, the limited and variable width of the benches would allow the block to increase its speed while falling, and possibly fall into the access road and spillway. However, it has to be mentioned that no significant rock falls have occurred to date but, if nothing was done, it was expected that falls may occur in the future, particularly in the spring when the ice thaws and releases blocks loosened by the ice.

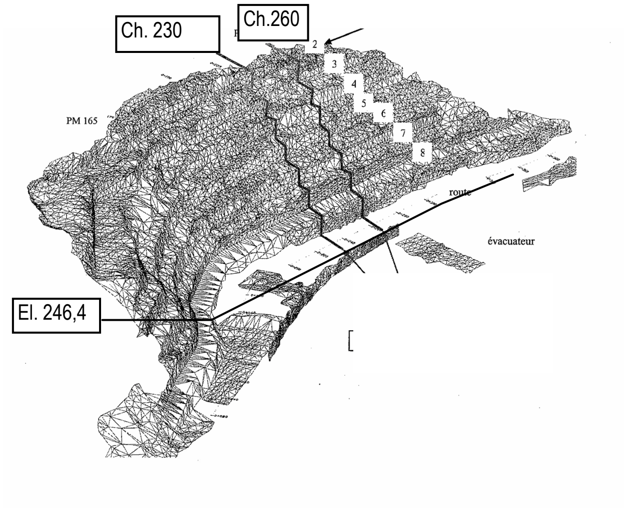


Figure 5. 3-D representation of the spillway rock cut.

4. ROCKFALL HAZARD

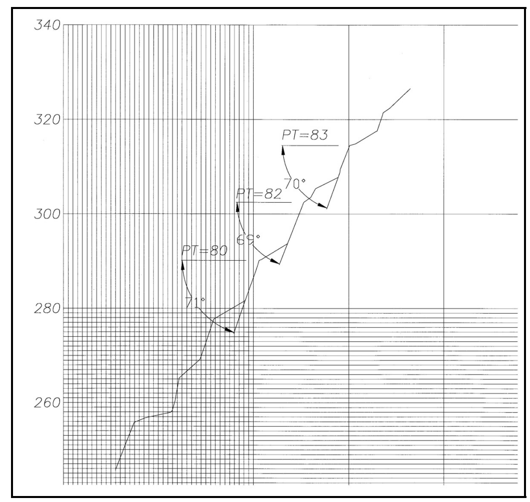
From the geology and the structural features determined by mapping of the slope face, it was assumed that the falling blocks will tend to have dimensions of 1 to 2 m because of the wide joint spacing, and would not break up significantly as they impact the face because of the high rock strength (anorthosite compressive strength ranging from 130 to 240 MPa).

As mentioned in the introduction, the main access road is located at the toe of the slope. Also, during the concreting of the spillway structure, many workers would be present in the spillway canal for the few months corresponding to the start of spring time. Protection from rock falls during and after construction had to be guaranteed.

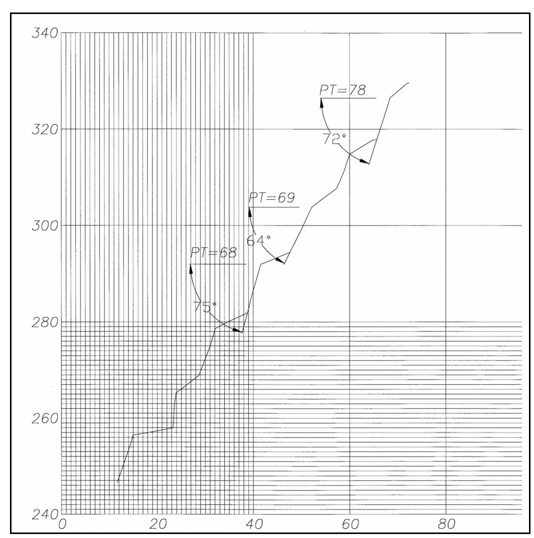
4.1 Rockfall Simulation

A commercially available computer program was used to model rock fall behaviour to help design the protective measures and the dimensions and location of catchment areas. The most important variables in this type of analysis, and the most difficult to define, are the coefficients of restitution (Rn: normal and Rt: tangential) of the slope materials. Nevertheless, preliminary simulations were performed with the software to estimate potential trajectories of falling rocks. Furthermore, one of the objectives was to assess if a falling block could reach the bottom of the spillway canal where the concreting work was to be carried out. It was found that if the default values provided in the program were used, the blocks seem to present unrealistic bounce heights and reached the canal. It was decided to proceed, on site, with real rolling rock tests which are described in more detail in the next section. Another purpose of the in situ rock fall tests was to calibrate and improve the reliability of the model.

The rolling rock tests revealed that the actual behaviour involved crushing of the falling rock at impact with the rock surface, and generation of spin. The actual falling rocks had relatively flat trajectories after each impact in comparison with the simulations.



a)



b)

Figure 6. Resulting profile of the spillway right bank; a) at Chainage 260 and b) at Chainage 230.

Using the surveyed impact points on the slope during the tests, it was possible by trial and error to find the best range of values for Rt and Rn to use for the Péribonka rock mass (Table 4).

The remaining parameters needed to run the simulations are the block shape, horizontal and vertical initial speed, roughness, angularity and angular speed. It was assumed that the blocks were of spherical shape with initial speed of 0,1 m/s; other parameters were not taken into account, except the angular speed which is automatically accounted for in the software.

Table 4. Parameter optimization with rock fall simulations

Slope Section	Parameter Range Rt	Rn	ϕ	Observations
230	0.85	0.35	30	Initial values - Unrealistic
	0.75-0.80	0.4-0.6	35	Rn and bounces too high
	0.82-0.88	0.23-0.27	35	Similar to in situ tests
	0.9-0.95	0.2	35	Excessive rebounds
260	0.85	0.35	30	Initial values - Unrealistic
	0.75-0.82	0.35-.05	35	Rn and bounces too high
	0.85-0.90	0.28-0.32	35	Similar to in situ tests
	0.92-0.95	0.25-0.33	35	Excessive rebounds

4.2 Rockfall in Situ Tests

The rock fall tests involved gently nudging boulders off the crest of the cut on Sections 230 m and 260 m. The dimensions of the blocks were measured and their approximate diameter and weight were calculated; the blocks were approximately cubic shaped with sharp edges, rather than tabular or columnar. The blocks were classified according to their dimensions – Class A: 1.5 to 2 m^3; Class B: 0.9 to 1.3 m^3 and Class C: 0.45 to 0.76 m^3. There were eight blocks in each class. Figure 7 shows a picture of few of the blocks.

Figure 7. Blocks selected for the in situ rolling rock test

The rock fall trajectories were observed as they crossed the road at the base of the cut; the locations of the last two impact points were surveyed. A film recording of each test was made with a digital camera, the high speed camera did not work as these tests were conducted during a cold winter day.

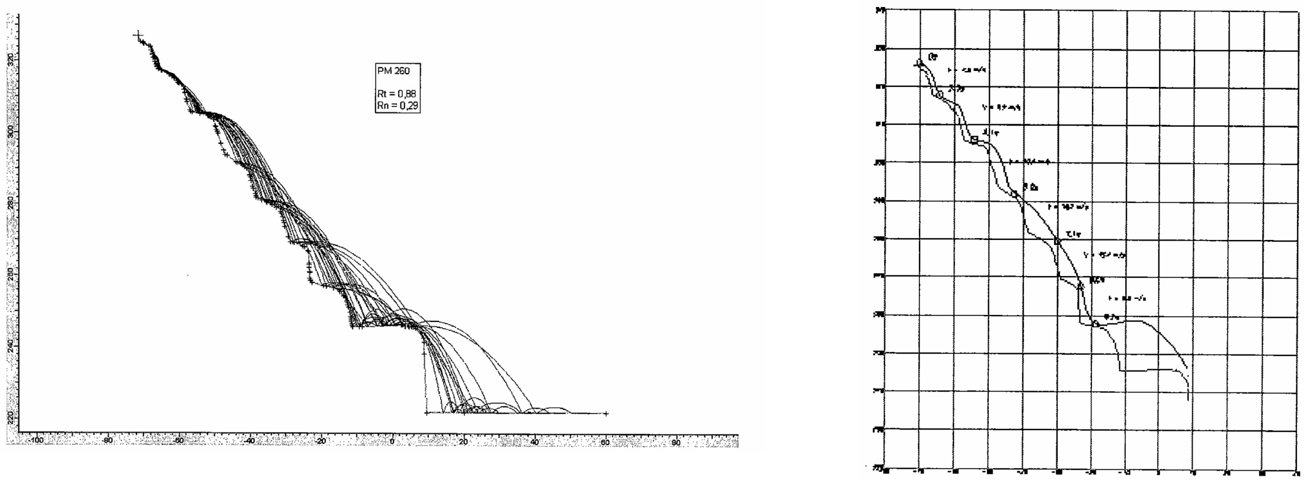
The following general observations were made from the tests:

- Snow accumulation on the benches absorbed some energy at the impact points and probably resulted in slower velocities and lower trajectories compared to impact on clean rock;
- Larger rocks (Classes A and B) had faster velocities and higher trajectories than smaller rocks (Class C) that often hung up on the upper benches;
- The rocks all fell from bench to bench and did not impact the intermediate faces. Towards the lower part of the cut, the rocks impacted either the lowest bench (Bench 8, elevation 246.4 m) or Bench 7 (elevation 256.8 m), then impacted the outer half of the road, and finally passed over the crest of the cut into the spillway channel. One rock fell directly from Bench 7 into the channel, just missing the edge of the road;
- The trajectories were between about 1 m and 3 m from the rock face following each impact on the benches, and after impacting the road, the trajectories were about 1 m above the road surface. This latter trajectory could be confirmed by observing the impacts with the 1 m high fence along the outer edge of the road;
- The observation that some rocks that reached the road did not impact Bench 8 showed that excavation of this bench, if considered, would not be effective in containing all rock falls. Also, the low trajectories after impact on the road meant that falls could be contained by a fence along the outer edge of the road

Figures 8a and 8b are presented side-by-side to compare the trajectories, at Chainage 260, resulting from a rock fall simulation (values of 0,88 and 0,29 for Rt and Rn respectively) and the surveyed trajectory of rock block #8A having a volume of approximately 1,6 m^3 and a weight of over 4 metric tons.

5. PROTECTION MEASURES

For this type of problem, few protection options are available. However, in the particular case of Péribonka, the time frame was a major issue as the spillway structure concreting had to start May 1st. After analyzing the different options, it was realized that the long term options required too much time to allow the timely implementation of concreting.



a) b)

Figure 8. Block trajectories from a) rock fall simulation and b) in situ testing

5.1 Options considered

Temporary protection measures were considered to be those that allowed installation prior to the end of April, but their effective life may be limited to about five years or less before significant maintenance is required. Long-term protection measures are considered to be those that will have an operational life approximately equal to that of the project, although maintenance will be required during this period. The options considered are listed below:

- Temporary protection options
  - Draped mesh on face
  - Ring net fence on road
- Long-term protection options
  - Mechanically stabilized earth (MSE) wall and fence on road
  - Hanging net on face
  - Rock face stabilization

Construction of the long-term systems, including design and procurement of materials, would have taken several months and not be complete before the start of spillway construction in May. Therefore, temporary measures were adopted until an eventual long-term system is put in place. For the time being, it was decided that a long term option design and implementation would only be required if it is found that the temporary option was not effective in containing rock falls.

5.2 Selected protection and installation

High strength draped mesh was installed over most of the slope to protect both the spillway construction and access road. Mesh sections of 30 m long by 3,5 m wide (105 m^2) as seen in Figure 9 were installed by Hydro-Quebec Construction team by following precise installation guidelines and preparatory works. Few parts of these guidelines are listed hereafter:

- All loose rock material and ice blocks were to be removed prior to the mesh installation;
- Installation of grouted anchors at the top of the slope on which mesh support cables were suspended (as illustrated in Figure 10);
- All suspended mesh sections were to be draped on the rock face without any pinning or bolting; only the top sections would be attached;
- Every horizontal and vertical joint would overlap.

Figure 9 High strength mesh roll to be installed

The draped mesh is used in combination with a catchment ditch located at the toe of the slope (Figure 11). As shown on Figure 11, the ditch includes a concrete safety barrier located few meters away from the rock face with sand and rockfill material placed as cushion material.

Careful observation of the condition of the installed mesh (Figure 12) were to be required during construction of the spillway and repairs carried as necessary, which may include stabilization of the rock slope behind the mesh. If the mesh performs satisfactorily, it may become the long-term protection system for the slope.

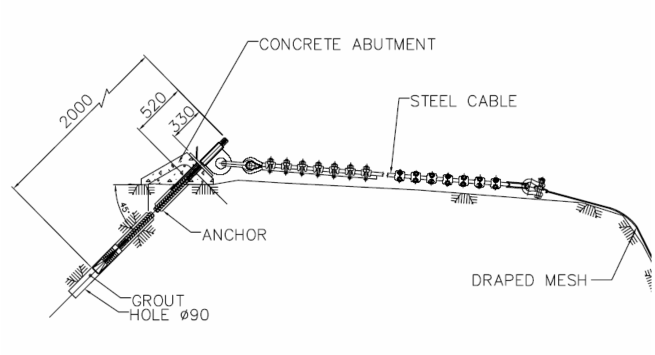


Figure 10. Typical installation of mesh anchoring system at top of cut.

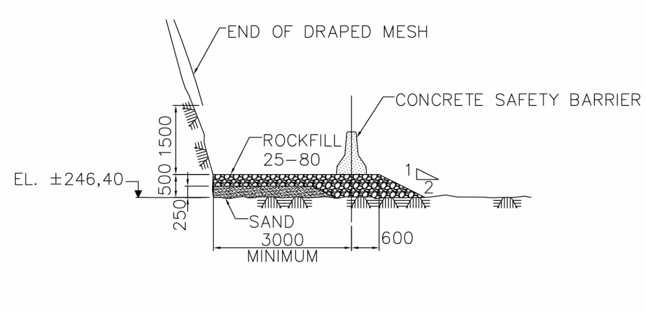


Figure 11. Catchments ditch at the toe of the slope

6. CONCLUSION AND RECOMMENDATIONS

A potential for rock falls and safety hazard existed from the rock cut above the spillway, with falls most likely to occur in the spring when the ice on the face thaws. Construction of the spillway was planned to start in early spring, so it was necessary that rock fall protection measures be in place by that time. It was considered acceptable that the protection measures installed over a two month period prior to spillway concreting could be temporary, i.e. have an operational life of perhaps two to five years, and that an alternative system that provides long-term protection be installed, if necessary, at a later date.

Figure 12. Mesh installation with crane on the slope face

A total of two temporary and three long-term protection options were considered for the cut. The selection of the most suitable options considered such factors as availability of material(s), the need for specialized construction personnel, construction by the end of April and the reliability of the system. It was decided that mesh draped on the face to contain the rock falls would meet these criteria. High strength mesh was to be used on the total surface of the cut above the spillway, to protect the workers and the road access below. The mesh was not to be attached to the face because at the time of installation there were ice and snow accumulations on the face and the objective was to guide falling rocks down the slope. Careful observation of the mesh condition and the face stability behind the mesh were required during spillway construction, and maintenance was to be carried out as required in the future. Since its installation two years ago, the mesh has performed well as the ice forming in the slope face has not damaged the mesh. Also, no blocks are reported to have reached the access road.

Regarding the numerical simulations of the rock falls, it was possible to determine a range of values that satisfactorily represent the rock mass response. The use of these values allowed simulations to be performed that were comparable to trajectories of actual blocks of rock rolling down the slope. If there is a need for the design of another protection measure in the future, these values will be helpful to conduct more realistic computer simulations.

Finally, concerning the rock cut design; it was observed that the potential for rock falls into the spillway cut relates primarily to the presence of benches on the face. It is very difficult to maintain full width, level benches in hard rock conditions because the blast damage occurs to the rock at sub-grade level. This damage occurs on the crest of the bench that is formed when the subsequent lift is excavated.

As a result of this condition, it is now usual practice for rock slopes on civil projects to excavate uniform faces, with bench widths limited to about 0.6 m as required for drill access to the final face (Wyllie & Mah, 2003). If benches are required on the cut, they are usually located at a vertical spacing of three to four excavation lifts, and have a minimum width of about 10 m. This width is sufficient to contain rock falls and provide access for clean up and maintenance equipment. In the present case, at hind sight, it seems that an optimization of the initial option (vertical bench) should have been considered.

These design considerations together with a good control on blasting methods and operations could certainly contribute to the quality of the final excavation and in providing a safer rock cut.

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