LOWER CHURCHILL PROJECT

Technical Memorandum
Slope Protection with Consideration for Ice Effects

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EXECUTIVE SUMMARY

For the project at Muskrat Falls, it is necessary to determine adequate design criteria and design philosophy for the slope protection on natural slopes and embankment dams to address the possible adverse effects of the ice loading. This report presents the hydraulic conditions prevailing at the site, the ice loading cases, the various scenarios of potential damage, and the approach that will be adopted to mitigate this situation.

During construction of the Muskrat Falls Hydroelectric Project, the river will be diverted through a partially constructed spillway and the level upstream from Muskrat Falls will be controlled such that a stable ice cover will be formed in winter; frazil ice generated upstream of Gull Lake will accumulate in the headpond and little frazil ice will be generated between Gull Lake and Muskrat Falls. The ice cover downstream of the falls will also consist of a solid "thermal" ice rather than an accumulation of frazil

As far as the water level downstream of the Falls is concerned, the regime will be governed by the flow passed by the Upper Churchill Project (a base load power generation operation) with some fluctuation if Muskrat Falls is operated as a peaking plant. The predicted levels in the reach downstream may fluctuate by 3 m within half a day, corresponding to discharge range between the maximum plant capacity of 2,660 m^3/s and the minimum discharge for environmental requirements of 540 m^3/s.

A literature review has been carried out in order to identify the state-of-the-art for the design of structures to resist the potential damaging effects of ice. In fact no rigorous procedure has been established and much is based on empirical approaches and past experience. Although the number of incidents of damage to the riprap on dams of hydroelectric generating stations is limited, certain observations as to the effect of ice have been noted and the approach to be adopted is indicated in the following.

The rip-rap will first be designed according to the usually accepted standards to resist wind generated waves. Considerations for ice attack will be complimentary.

Relatively shallow slopes (1V:3H) are preferred for limiting the damage due to ice run-up though the distance up the slope on which protection is required may be greater than for steeper slopes.

Well graded riprap, placed to create a smooth surface with no protruding stones will better resist plucking action.

To resist ice plucking of the rip-rap during fluctuations of the water level, the thickness of the rip-rap, which is a function of the stone sizes, should be at least equivalent to the anticipated thermal ice cover thickness.

The accumulation of frazil ice or ice debris may provide a certain degree of protection from the impact of a solid ice sheet.

The anticipated fluctuations of the reservoir and downstream river levels should be determined and included in the analysis of potential ice damage.

As several uncertainties have to be accounted for, there is a choice to be made between a conservative approach requiring heavier, thicker rip-rap and the alternative of a rational but not overly conservative design with the prevision for access should repair works be required in the future. Consideration needs to be given to the possible quarry sites to establish the degree of difficulty that would be associated with providing heavier rip-rap.

1 INTRODUCTION

In the course of the studies dealing with the hydraulic conditions at Muskrat Falls, a question was raised concerning the appropriate design criteria and design philosophy for the slope protection on natural slopes and embankment dams to address the possible adverse effects of the ice loading. This report presents the hydraulic conditions prevailing at the site, the ice loading cases, the various scenarios of potential damage, and the approach that will be adopted to mitigate this situation.

2 HYDRAULIC REGIME

In the natural state, frazil ice generated in the long sections of rapids upstream from Muskrat Falls, accumulates in the slower, wider section of the river immediately downstream of the lower falls. This accumulation can reach several meters in thickness and constrict the flow in the river such that water levels rise by 2 m or more above equivalent ice free levels (Ref. 12).

The downstream rating curves shown in Figure 2-1 illustrate the hydraulic regime in question. It has been found that there are three different discharge-level relationships on the Churchill River below Muskrat Falls based on the river ice conditions (Ref. 13). Therefore, depending on the period of the year, for the same river discharge there could be three different water levels as shown in Figure 2-1. They correspond to the open water conditions approximately between May and November, the freeze-up conditions in December and the winter conditions between December and April.

During construction of the Muskrat Falls Hydroelectric Project, the river will be diverted through a partially constructed spillway and the level upstream from Muskrat Falls will be controlled such that a stable ice cover will be formed in winter; frazil ice generated upstream of Gull Lake will accumulate in the headpond and little frazil ice will be generated between Gull Lake and Muskrat Falls. The ice cover downstream of the falls will also consist of a solid "thermal" ice rather than an accumulation of frazil (Ref. 12).

This situation will be continued for the normal operation of the project, albeit at upstream water levels above the temporary diversion levels. That is at the full supply level of El. 39 m rather than El. 25 m which applies for the diversion period. The fluctuation of the reservoir level upstream of Muskrat Falls is expected to be at maximum 0.5 m per day or less until the commissioning of Gull Island.[1]

[1] Muskrat Falls should be operated as a run-of-river plant after the commissioning of Gull Island.

As far as the water level downstream of the Falls is concerned, the regime will be governed by the flow passed by the Upper Churchill Project (a base load power generation operation) with some fluctuation if Muskrat Falls is operated as a peaking plant. The predicted levels in the reach downstream may fluctuate by 3 m within half a day, corresponding to discharge range between the maximum plant capacity of 2,660 m^3/s and the minimum discharge for environmental requirements of 540 m^3/s.

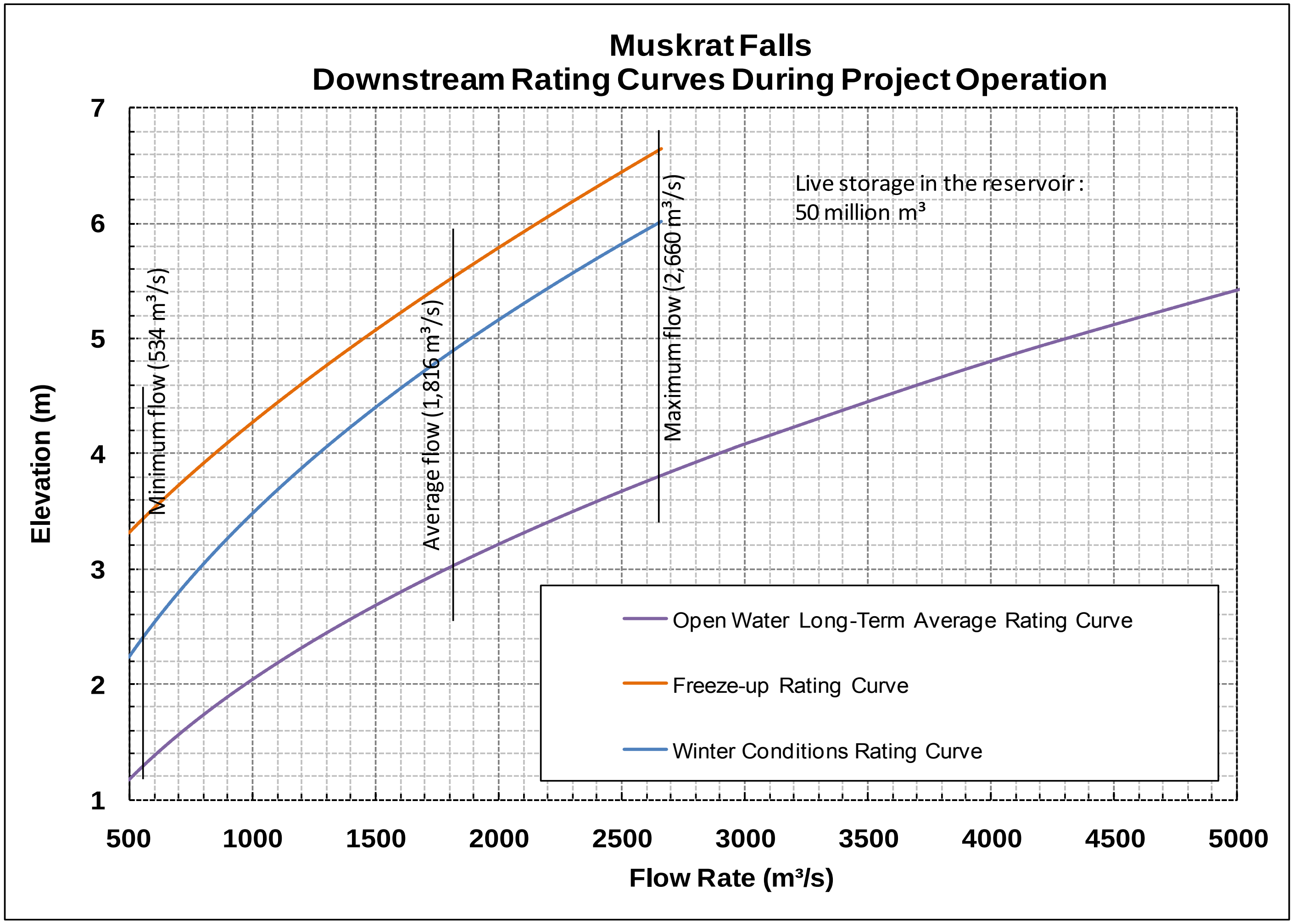

Figure 2-1: Downstream Rating Curves

3 ANTICIPATED POTENTIAL ISSUES

Along the banks of the Churchill River, including the North Spur, which will become an intrinsic part of the Muskrat Falls water retention structures, landslides constitute one of the more obvious geomorphologic features. The landslides are caused by progressive erosion of the toe of the natural slopes but can be triggered by events such as rapid water level fluctuations at spring breakup. The long term preservation of the North Spur will include works such as riprap protection along the shore line downstream of Muskrat Falls as well as in the drawdown zone in the reservoir upstream.

This riprap will be designed for wave attack based on the project design wind velocities. However, rip-rap slope protection can also be subject to degradation caused by ice. This can be from three actions of the ice:

1. Mass movement: The thrust from the ice sheet due to wind or current tending to cause a slide or shear along a horizontal surface through an embankment or jetty.
2. Gouging or Shoving: The ice sheet moving along the shoreline gouges riprap stones from their normal location, or the ice rides up on the slope and displaces the riprap stones.
3. Ice plucking: The ice sheet grips individual stones or groups of riprap stones and, with rising water (and ice) levels, lifts the stones from their bedding. Alternatively, with a falling reservoir level a slab of ice can act as a cantilever and pull the stone out of place.

Any or a combination of the above phenomena could cause damage to the protection of the natural slopes at Muskrat Falls or, indeed, of the south embankment dam.

4 SOURCES OF INFORMATION

The ice loading on structures has been the subject of many publications. Several deal with the load on rigid structures due to dynamic forces from moving ice sheets or static forces from ice sheets already in contact with structures and pushed by wind or current, in addition to the forces due to the thermal expansion of the ice sheet. The effect of ice on riprap occupies less space in the literature.

The references listed below were consulted in the preparation of this report.

Carter, D., 2003, *Guide pratique pour le calcul des forces exercées par la glace*, Study report prepared for Hydro-Québec

Croasdale, K.R. and Marcellus, R.W., 1977, *Ice and Wave Action on Artificial Islands in the Beaufort Sea*

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SEBJ, 1997, *Practical Guide, Riprap Sizing*

Sodhi, D.S., Borland, S.L. and Stanley, J.M., 1996, *Ice Action on Riprap, Small Scale Tests*, CRREL Report 96-12

In addition, a request for information was sent out in April 2012 to various Canadian utilities. Responses were obtained from:

- BC Hydro;
- Hydro-Quebec;
- Manitoba Hydro; and
- Ontario Power Generation.

However, little new information was derived from these enquiries.

A contribution to the discussion was received from Northwest Hydraulic Consultants in the form of a memorandum dated June 30th, 2013 and entitled “Muskrat Falls Hydro Project, Review of ice process and Preliminary Ice Force Evaluation”, this memorandum is presented in Appendix A.

5 SUMMARY OF PAST EXPERIENCE AND DESIGN APPROACHES

Croasdale et al. (1977) describe the design and exploitation of artificial islands in the Beaufort Sea for drilling platforms. The potential failure mode type 1, mentioned above, that of shearing failure through the island, was recognized as a distinct possibility, though no cases of damage were reported. Cutting stress relief slots was seen as a mitigative measure.

Croasdale et al. (1988) gives additional information on design approaches for the offshore artificial islands in the Beaufort Sea to reduce the effect of ice ride-up which could damage the installations on the artificial islands. A smooth shallow angle slope increases ride-up potential whereas breaks in the slope, and steeper slopes reduce the risk. Interestingly, submerged beaches are favoured as early winter ice rubble is grounded on such beaches and provides a barrier for further encroachment of the ice sheet.

Doyle (1988) describes the damage caused by an early break-up and high water flows carrying thermally un-deteriorated ice in the Nicola river system in B.C. This is an example of the type 2 damage.

There have been limited documented cases of ice plucking of rip-rap and even fewer of damage due to other mechanisms. The report prepared by Matheson (1988) for the Canadian Electrical Association is an assembly of information gathered from questionnaires sent to a variety of Canadian and International Utilities together with inspections of several dams in the Prairie Provinces of Canada. Two cases of riprap stones being plucked from a protective zone by rising water levels are cited. One is from the Grand Rapids Generating Station in Manitoba where a 1.6 m rise in the forebay elevation in February 1972 plucked stones from the riprap. The other reference concerns the required repairs along a coastal embankment in Cook Inlet, Alaska, where winter ice thicknesses of around 0.7 m combined with tidal fluctuations of more than 12 m caused damage that required repair work with stones of up to 2,700 kg.

At Grand Rapids in Manitoba, there were 1974 reports of ice shoving causing damage to riprap a meter or more above the full supply level.

The ICOLD Bulletin No. 105 of 1996, states that there is no fundamental approach to designing riprap to resist ice action. However, the mechanisms of damage are described, including:

- Jacking of riprap stones by ice lenses formed from water seeping out of the embankment during a winter period of falling reservoir levels;
- Cantilevering of slabs of ice as the reservoir drops;
- Sliding of masses of ice down relatively steep slopes (1V:1.75H) as the water level drops;
- Lifting (plucking) of stones bonded to ice as it rises with increasing reservoir levels;
- Shoving of small stones <0.45 m in Manitoba and in China.

In general, a thick layer of dumped rockfill is judged to be more appropriate than a single layer of hand placed riprap or of concrete slabs when ice action is being considered.

Sodhi et al. (1996) conducted small scale experiments and drew the following conclusions:

- Smooth riprap surfaces are preferred for resisting ice which is contradictory to the energy dissipating rough surfaces when wave run-up is being considered;
- To resist ice shoving, on a 1V:3H slope the stone size should be 2x the ice thickness and 3x for steeper slopes of say 1V:1.5H. However, the authors also stated that it may be more cost effective to carry out repairs than to design to these requirements.

MacDonald (1988) also stated that smooth slopes of graded riprap were better at resisting ice than rough, uniformly graded riprap. In either case, stones should be well keyed.

In the 1997 *Practical Guide to Riprap Sizing*, published by the James Bay Energy Corporation (SEBJ), the aspect of ice effects is only briefly mentioned and it is concluded that the rather severe design hypotheses for the Northern Quebec reservoirs, assumed in the 1997 riprap design for wave attack, ensures adequate protection against ice forces. Prior to this publication, a W_{50} of 454 kg (D_{50} of 0.65 m) had been adopted on several projects such as Manic 3, Outardes 2 and La Grande 2, in order to consider ice effects.

The guide prepared by Carter for Hydro-Québec, in 2003, is a more recent work. The recommendations for reservoirs exhibiting a tendency for a possible sudden rise during the winter period included a D_{50} of at least the anticipated ice thickness. This is likely to be the governing equation for reservoirs with a fluctuating level, as the formulae for floating ice sheets at spring break-up lead to lesser values. However, for reservoirs likely to undergo a progressive drawdown throughout the winter, the loading is judged to be less severe and stone sizes of 0.75 times the ice thickness is quoted but no mathematical justification is presented. It is assumed simply that the weight of ice has a favourable influence. This does not seem to cover the cantilever mechanism, though it must be said that the required combination of stone position, slope angle and ice thickness to form a cantilever, presents a less probable scenario than the more simple adhesion of stones to a rising ice sheet. There is therefore some logic to the acceptance of a less stringent requirement for the case of reservoir level lowering as compared to a rising level.

The review carried out by Northwest Hydraulic Consultants (2013) brought to light some areas where the interpretation of experimental results, including some of the work cited above, may be conservative. For example, when examining the potential for stones to be plucked out of the rip-rap by a rising ice sheet, it is pointed out that the ice bonded to the stone also penetrates the full thickness of the rip-rap and thus an individual stone is not solicited but the entire thickness of the rip-rap stones. As the rip-rap layer thickness is at least twice the D_{50}, then the stone size to resist plucking is about one half of that needed if the individual stone is considered. Rip-rap thickness can also be specified as being 2.5 times the smallest stone size for uniformly graded rip-rap.

Similarly, for the gouging or shoving mechanism, if failure of the ice sheet in bending is considered, as the work by Sodhi apparently suggests, rather than crushing, then the forces transmitted by a moving ice sheet will be less.

The rip-rap sizing has to consider all scenarios of solicitation. This is site specific and wind and waves may govern in one location whereas ice induced loading may be critical elsewhere.

6 APPLICATION TO MUSKRAT FALLS

From the above mentioned data sources, the following points can be made:

1. Flatter slopes, of say 1V:3H, reduce the incidence of ice damage;
2. To resist plucking, the rip-rap thickness which is related to the average or minimum stone sizes should be greater than the maximum ice thickness;
3. To resist ice shoving, a D_{50} of about 0.30 m would be adequate for the slope angles envisaged;
4. The surface should be smooth and all stones well keyed in.

As far as the quality of rock riprap materials are concerned, sedimentary rocks are definitely inferior to igneous parent rock, but there is no universal series of tests to specify the rock quality. In general, high density, low absorption and good sulphate soundness test results are required but the commonly used Los Angeles abrasion test does not correlate well with freeze-thaw resistance.

The recommended tests with acceptance criteria included in the CEA (Matheson 1988) document are:

Table 6-1: Recommended Laboratory Testing for Riprap and Acceptance Criteria (Ref. 8)

Test	Rock Type	Method	Acceptance Criteria	Comment
Specific Gravity	All	ASTM C127 CSA A23.2-12A	$\geq$ 2.60	-
Absorption	All	ASTM C127 CSA A23.2-12A	$\leq$ 2.0	Primarily applicable to igneous & metamorphic rocks
Sulfate Soundness	Sedimentary & meta-morphic	ASTM C88 CSA A23.2-9A	$\leq$ 10% loss	-
Los Angeles Abrasion	Sedimentary & metamorphic	ASTM C535 CSA A23.2-17A	$\leq$ 30% loss	-
Freeze-thaw	Sedimentary & bedded metamorphic	CRD C144	$\leq$ 1% See (1)	Tests to be conducted on slabs per-pendicular to bedding

Notes: (1) Durability assessed on qualitative basis, if weight loss after 20 cycles is $\leq$ 1% rock is considered durable. For breakdown > 1%, field spacing of features along which break-down occurs should be assessed as with wide spacing rock may be acceptable.

As there are only a limited number of documented cases of damage to structures from ice action, the design procedures are not rigorous. Some insight from precedent can be gained but the possibility that maintenance will be required should be considered.

For the downstream toe of the North Spur, wave action may not be the determining factor in the riprap design and ice action is to be considered. However, the need for heavy rip-rap is not envisaged. Easy access for construction equipment should be part of the design. A slope of 1V:3H in this area would not entail significantly greater volumes of materials.

For the Muskrat Falls reservoir, the slope protection on the upstream side of the North Spur and on the South Rockfill Dam will be designed primarily for wave action. However, some fluctuation of the reservoir water level is anticipated for spring flood events exceeding 1:1,000-year floods. A stable thermal ice cover will form on the reservoir and the possibility of stones being plucked from the riprap has to be considered. However, the remoteness of this flood event combined with the spring ice conditions characterized with softer ice may not require a riprap design for ice conditions.

If the reservoir fluctuations are planned for a narrow range, it may be of interest to incorporate larger riprap only in the potentially affected band on the slopes and/or to incorporate an access berm at an appropriate elevation immediately above the affected zone.

7 CONCLUSION

The literature contains several references to slope protection design for ice. However, many of these concern the works constructed in the Beaufort Sea for the oil and gas industry. The number of incidents of damage to the riprap on dams of hydroelectric generating stations is limited. Some consideration is given to the potential for damage but a rigorous design procedure does not exist.

The rip-rap will first be designed according to the usually accepted standards to resist wind generated waves. Considerations for ice attack will be complimentary.

Relatively shallow slopes (1V:3H) are preferred for limiting the damage due to ice run-up though the distance up the slope on which protection is required may be greater than for steeper slopes.

Well graded riprap, placed to create a smooth surface with no protruding stones will better resist plucking action.

Stone sizes resisting plucking should be such that the thickness of the rip-rap layer nominally twice the D_{50} size or 2.5 times D_{min} is at least equivalent to the anticipated thermal ice cover thickness.

The accumulation of frazil ice or ice debris may provide a certain degree of protection from the impact of a solid ice sheet.

The anticipated fluctuations of the reservoir and downstream river levels should be determined and included in the analysis of potential ice damage.

As several uncertainties have to be accounted for, there is a choice to be made between a conservative approach requiring heavier, thicker rip-rap and the alternative of a rational but not overly conservative design with the prevision for access should repair works be required in the future. Consideration needs to be given to the possible quarry sites to establish the degree of difficulty that would be associated with providing heavier rip-rap.

APPENDIX A

Muskrat Falls Hydro Project

Review of Ice process and

Preliminary Ice Force Evaluation

By

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memorandum

To:	Daniel Damov, Eng., P. Eng. Senior Engineer Hydrology/Hydraulics SNC-Lavalin Inc.	**Date:**	June 30, 2013
From:	**David Andres, P. Eng.**	**No. Pages:**	32 with figures
		Project No.:	100071

Re: **Muskrat Falls Hydro Project – Review of Ice process and Preliminary Ice Force Evaluation**

INTRODUCTION

In addition to the changes to the mesoscale ice characteristics that will arise out of the Muskrat Falls Project, there are a number of ice-related concerns in the vicinity of the facility itself. Upstream of the structure, within the reservoir the main concerns are (i) the effects of ice on the upstream face of the South Dam, (ii) the effects of ice on the upstream side of the North Spur, and (iii) the performance of the ice/log boom that will span the reservoir from a point on the flank of Innu Spirit Mountain (a rock knoll at the north end of the North RCC Dam) to the south shore of the reservoir. Downstream of the facility, the main concern is the effect of ice on the stability of the shoreline protection along the downstream side of the North Spur.

This report provides a preliminary description of the ice conditions in the reservoir. It also describes the mechanisms that will lead to the action of the ice on the ice boom and the rock riprap on the various embankments, and provides criteria for the determination of subsequent ice forces. The variables that need to be defined to calculate ice forces include ice thickness, ice strength, floe sizes, and floe velocity. Ice thickness can be determined from modelling ice cover growth and deterioration. The others can be determined from interpretation of the design codes, from the planform characteristics of the reservoir/tailrace, embankment characteristics, and from practical considerations of the design meteorological conditions. Some of these design choices are somewhat subjective, are based on experience, and are open to debate.

The overall design process ultimately will be iterative because the ice forces are a function of both the ice characteristics and the structure characteristics (rock sizes, pontoon diameters, etc), which are still being finalized. From this perspective, the contents of this report are not intended to be viewed as final designs. Rather, the report is meant to provide an independent and perhaps alternative view of the important ice-related design issues. Nevertheless, this report provides a preliminary evaluation of the ice conditions and their effects on salient structures to be used as a framework for future discussion, design parameter optimization, and more rigorous study in areas where there may be disagreement on potential post-regulation outcomes.

The report contains the following.

1. A brief discussion of the facility, reservoir characteristics, study area, the winter hydrology, and climate.
2. A discussion of the general ice processes and ice-related issues.
3. A description of the salient ice forces that could affect the structures.
4. A summary of expected ice forces for the current design of the embankment.
5. A summary of the expected ice actions on the log/ice boom and the corresponding forces that it could experience.

CHARACTERISTICS OF THE MUSKRAT FALLS PROJECT

General

The central elements of the Muskrat Falls Project consists of a power intake, a tailrace channel, and a gated spillway. The intake is flanked by the South Dam at the south end of the facility and the gated spillway is flanked by the longer North RCC Dam that contains an overflow spillway. The North RCC Dam butts up against Innu Spirit Mountain that anchors the north end of the facility. Innu Spirit Mountain is separated from the main part of the north valley wall by the North Spur, which is a very thick deposit of consolidated glacial material.

The expected full supply level (FSL) or normal operating level (since it is essentially a run of river facility) of the reservoir is 39 m – equivalent to a water depth of about 30 m just upstream of the facility. The reservoir will extend upstream to the tailrace of the Gull Island Project – a distance of about 100 km. The reservoir volume will be 1,500,000 dam^3 (or 1500 km^3) at FSL and it will have a surface area of about 100 km^2. Overall, the mean reservoir depth will be 15 m with an average cross sectional area of about 25,000 m^2. The flow into the reservoir is controlled by operations at Churchill Falls. The average winter inflow is about 1800 m^3/s but it could vary between 1000 and 2000 m^3/s. The typical current velocity at the lower end of the reservoir will be about 0.060 m/s.

Powerhouse flows during the winter will vary diurnally in response to changing power requirements and varying inflows into the reservoir. The range in flows will be between 500 m^3/s and 2600 m^3/s and it is expected that the flows could be ramped up and down between those limits over periods of one or two hours. Therefore, water levels in the tailrace will vary by about four metres over a period of a day from a minimum water level of 2.2 m to a maximum of 6.0 m (assuming a “winter conditions” rating curve). The flow pattern and subsequent velocities will be highly two dimensional in and immediately downstream of the tailrace. Owing to the presence of a large "scour" hole where a hanging dam currently forms, the average velocity will be very low. From a two dimensional perspective, a large eddy will likely form adjacent to the North Spur. In plan, the eddy will have a surface areas of about 500,000 m^2 and its rotation will be driven by the transfer of momentum from the high velocity "jet" exiting the tailrace channel. It is not clear exactly how this eddy will behave (as one large one or a number of smaller ones) but it is likely that floating matter that collects within the eddy will stay in the eddy over the period that the powerhouse flows remain steady.

Specific Structures

The four main structures/components that potentially will be affected by ice are the upstream face of the South Dam, the embankment on the upstream shore of the North Spur, the embankment on the downstream shore of the North Spur, and the upstream ice/log boom. ***Table 1*** summarizes the physical characteristics of the three rock structures. The boom will be located in the vicinity of km 44.0. The characteristics of the boom have not yet been finalized, but the boom likely will be about 600 m long, with likely span lengths of no more than 100 m – i.e. the boom would be composed of six spans each anchored to the bed by a set of anchor cables. Each span would contain about 15 pontoons if a conventional pontoon length of 8 m is adopted.

Table 1 Adopted riprap characteristics at selected structures [1]

Parameter	South Dam	Upstream Shore of North Spur	Downstream Shore of North Spur
Side slope (xH:1V)	1.7	3.1 to 4.0	3.0
Bottom elevation (m)	36.6	35.8	1.3
Top elevation (m)	46.3	42.9	7.5
Rock diameter (1), D_{50} (mm)	510	590	470
Thickness of riprap layer (2) (mm)	900	1500	1800

(1) Based on wind and wave analyses.
(2) Based on a variety of considerations including wind and wave analyses.

[1] SNC-Lavalin. 2012. Riprap Design for Wind-Generated Waves, Table 5-8. Report submitted to Nalcor Energy. Nalcor Document # MFA-SN-CD-0000-CV-RP-0006-01.

CLIMATE CONDITIONS

The study area is located at a latitude of 53.25 °, just south and west of Goose Bay, Labrador. The climate station at Happy Valley-Goose Bay (station #8501900, Goose A, NFLD) provides a reasonable record of the regional climate – daily air temperature, precipitation, snow on ground, bright sunshine, and wind – that is germane to quantifying the ice characteristics.

Figures 1 to ***3*** show the salient normal monthly climate characteristics for the period of 1971 to 2000. The winter tends to span the months of November through to the middle of April and the typical average winter temperature is -12.5 ^{0}C (***Figure 1***). Snow can occur in every month except the months of July and August (***Figure 2***). Between November and March, total snowfall is about 360 cm. This contributes to a snow depth on ground in March of only about 80 cm due to the combined effects of compaction and ablation. General melting of the snowpack occurs over a period of about two months between mid-March and mid-May.

As expected, the minimum solar radiation occurs in December – with a daily average of about 25 W/m^2 (***Figure 3***) or 2.2 MJ/m^2/day. This is equivalent to an air temperature offset of about 1 ^{0}C. In April, the solar radiation increases to about 150 W/m^2 (12.8 MJ/m^2/day) which is equivalent to about 6 ^{0}C in additional air temperature. In terms of energy budget, solar radiation does not play a significant role in the dead of winter but it is very important in the spring.

Aside from the obvious affects of air temperature, snowfall, and solar radiation on the water temperatures and the thickness and quality of the ice, the wind is the most important climatic factor on the manifestation of ice forces on the structures. The historical measured winter wind conditions at Goose A indicate the winds are from the west and the west southwest for about 30 percent of the time and from the west northwest, northwest, and north northwest for about 18 percent of the time (***Figure 4.1***). The mean annual wind speed also follows more or less the same pattern with the highest winds originating from the west southwest to north northwest directions (***Figure 4.2***). Historically, the mean all-direction wind speed is only about 4.4 m/s, but the maximum hourly wind speeds range from 15 to 25 m/s (***Figure 4.3***).

Frequency analyses of wind speeds at the climate station has been undertaken by SNC-Lavalin [2] and the results are reproduced herein. The fetches are all quite short and hourly wind speeds would be the most germane to define the design wind speed. ***Table 2*** summarizes the wind speed statistics for the salient directions and ***Table 3*** provides the design winter wind speeds for the structures that may be affected by ice forces.

[2] SNC-Lavalin. 2012. Wind-Generated Wave Study, Table 4-1. Report submitted to Nalcor Energy. Nalcor Document # MFA-SN-CD-0000-CV-RP-0010-01

Table 2 Salient fetch directions and wind speed statistics at Goose A

Structure	Critical Direction	Hourly Wind Speed (m/s)			
		2-year	20-year	100-year	1000-year
Ice boom	NW (315 degrees from north)	12.7	18.0	21.3	26.3
South Dam	W to NW (300 degrees from north)	12.5	16.1	18.3	21.7
Upstream Shore of North Spur	WSW to WNW (270 degrees from north)	15.5	21.4	25.0	30.6
Downstream Shore of North Spur	ENE to ESE (90 degrees from north)	7.5	10.5	12.5	15.6

A design wind speed of between 20 and 25 m/s has been adopted for the structures affected by winds from the northwest and the west respectively. A wind speed of 15 m/s has been adopted for the structures subjected to winds from the east and southeast to account for both the lower wind speeds and reduced likelihood of experiencing wind from that direction. These values are less than the maximum wind speed on record, but have return periods of about 100 years – somewhat conservative but a design standard that reflects that applied by SNC-Lavalin to the analysis of wave forces on rock riprap. While there is a convention to increase wind speeds measured over land when applying those speeds to open water, this would not be required herein because the wind speeds are measured over open terrain that reflects conditions over water and the reservoir itself is quite narrow. In fact, in this case, such adjustments may produce overly conservative estimates of design wind speeds.

Table 3 Salient fetch directions and mean wind speeds in the reservoir

Structure	Critical Wind Direction (Degrees)	Critical Wind Direction	Mean Winter Wind Speed at Goose A (m/s)	Adopted Design Wind Speed (m/s)
Ice boom	315	NW	5.1	20
South Dam	300	W to NW	5.1	20
Upstream Shore of North Spur	270	WSW to WNW	5.4	25
Downstream Shore of North Spur	90	ENE to ESE	5.3	15

ICE CHARACTERISTICS

The main challenges are to characterize the ice conditions at the site during (i) freeze-up when ice is forming in or flowing into the reservoir, (ii) throughout the winter period, and (iii) during spring when the ice cover begins to lose its integrity due to surface melting and has the potential to be mobilized by wind. Ice conditions in the reservoir will be dramatically different than in the tailrace due to the different flow conditions.

Reservoir

Freeze-up Period

Experience suggests that – similar to ice formation processes on northern lakes – an ice cover can form only after the water column cools to 4 ^{0}C and during a period when winds are low and the air temperature is low enough to provide sufficient energy loss to the atmosphere to offset both the latent heat produced by the formation of ice and the heat supplied by conduction from the warm water below the water surface. Ice will first form along the shoreline and in the smaller bays and eventually extend out into the main parts of the reservoir as the wind conditions become more favourable.

Hatch [3] has carried out a thermal analysis of the reservoir. The analysis suggests that the ice cover will form on the reservoir at about the same time as the water temperature in the river upstream of the reservoir cools to near 0 ^{0}C. The ice cover likely will form according to the following processes.

1. Water temperatures upstream of the reservoir will more or less track the air temperatures. Water in the reservoir will cool in response to both heat loss from the water surface and the displacement of the warmer reservoir water by the inflow of cooler water from upstream. For a reservoir volume of 1,500,000 dam^3 and an average inflow of 1800 m^3/s, it would take about 10 days to displace all the water in the reservoir.
2. Cooling at the water surface of the reservoir will promote the formation of shore ice and possibly sheet ice. Shore ice certainly will form along the shore of the reservoir, but it is unlikely sufficient sheet ice will develop to form a stable ice cover because of the adverse effects of the wind and the currents. Winds and the resulting waves will tend to mix the surface of the reservoir and prevent the necessary stratification to allow the formation of significant amounts of either shore ice or sheet ice.
3. Ice will form in the river when the air temperature decreases to below freezing. This ice will enter the reservoir, add to the sheet ice that may have formed on the reservoir, be transported downstream by the currents, and accumulate against both the boom and the North Spur. The ice will be acted up by both the current and the wind. In fact, the wind

[3] Hatch Engineering, 2013. Muskrat Falls Ice Study - 2013 Update. Report prepared for Nalcor Energy - Lower Churchill Project. Report number MFA-HE-CD-2000-CV-RP-0006-01.

would likely have a somewhat greater effect on the stability of the ice than the current. If the ice is driven past the ice boom it would accumulate against the South Dam and the power plant and spillways. Consequently, ice forces could develop from the action of relatively thin floes composed of relatively strong ice.

From a design perspective, the most favourable situation would be the development of a sheet ice cover over one or two nights when the entire reservoir would develop an ice cover more or less instantaneously. Under this circumstance, the transfer of forces from the wind to the ice and then the structures (shoreline, ice boom, North Spur, etc) would be distributed along the entire shore line of the reservoir. However, this would require very cold temperatures with virtually no wind and would be unlikely to happen.

Eventually, the leading edge of the ice cover would advance upstream to cover the entire surface of the reservoir. The length of time for this to occur would depend on how much ice was forming in the reservoir and how much ice was being transported into the reservoir from upstream. From consideration of the surface area of the reservoir – say 100,000,000 m^2 – and how much ice could be transported into the reservoir it would take between 5 and 10 days for an ice cover to form over the entire reservoir if the air temperature remained significantly below freezing for that long of a time period.

From considerations of the mean velocity in the reservoir (***Figure 5***) it appears that it would take about 10 days for an ice floe to travel the length of the reservoir in the absence of wind. Assuming a nominal heat transfer coefficient of 20 W/m2/^{0}C, during that time the thickness of the ice floe could reach 0.35 m at air temperatures in the vicinity of -15 ^{0}C. It is unlikely that this would occur without skim ice first forming on the reservoir and accumulating against both boom and the faces of the structures. It is not immediately obvious what the thickness of the skim ice would be at the point when a stable ice cover would form but it is likely it would be in the range of 0.10 to 0.20 m (one to two days of growth after its initial appearance on the water surface). The ice would be formed under relatively cold conditions and could be considered to be "strong" ice.

Winter Period

Once an ice cover has formed on the reservoir, it will thicken in response to the severity of the winter temperatures and how much snow accumulates on the ice cover. Hatch simulated the expected ice thickness for a historical period of record. Typically, the ice cover would reach its maximum thickness of between 0.52 and 1.2 m in early April, begin deteriorating in late April, and likely be off the reservoir by the end of May. The analysis behind the ice growth simulations is quite rigorous and provided good estimates of the expected ice thickness that could develop throughout the winter period.

The main ice-related issues during the winter would be thermal forces and wind-related forces. The latter would not be expected to be too severe since the ice cover would be intact and in contact more or less with the entire shoreline, thus absorbing any possible wind-related perturbations to the ice cover. The former could be problematical along the faces of vertical structures, but these structures would have been designed for such events. These thermal forces are not within the scope of this review.

Breakup Period

With the return to above freezing temperatures in April and in combination with increased solar radiation, the snow cover begins to melt and when the snow is gone the ice cover begins to deteriorate. The deterioration takes two forms – (i) thinning due to absorption of solar radiation at the ice surface and by temperature-related heat transfer processes (convection and long wave radiation) and (ii) internal deterioration due to transmission of solar radiation into the body of the ice. The internal deterioration reduces the strength of the ice cover. While the Hatch simulation of the ice growth is defensible, the simulated ice deterioration rates that would occur in April and May, that would lead to the destabilization of the ice cover, and that would be the most germane to the estimates of ice forces are not so sophisticated. Nevertheless, the analysis provides a good starting point for looking at the expected ice thickness. The range in the thickness of the ice sheet when it is first likely to be mobilized would be 0.5 to 1.0 m (***Figure 6***)

Initially, the ice near the shore would melt away. This would create an opening between the intact ice sheet and the shoreline and provide room for the ice cover to move in response to winds and currents. Eventually, so much open water develops that, if and when a strong wind occurs, the intact sheet ice can be mobilized and driven against the shore, or any protruding structure. The subsequent ice forces would develop from the interaction of thick floes, composed of relatively weak ice, with the shoreline. Either the kinetic energy of the floe or its strength would limit the magnitude of the interaction.

Tailrace

Ice conditions in the tailrace are much less complex than those in the reservoir. The water temperature in the tailrace will be at least 0 ^{0}C and the Hatch analysis suggests that perhaps it could be as high as 0.2 ^{0}C due to the heat produced by the turbine inefficiencies.[4] As the highly turbulent flow leaves the tailrace, it will cool at a rate that depends on the water depth and the air temperature. Ice will not form at least until the nucleation temperature is reached – say -0.05 ^{0}C – at which point frazil will develop throughout the water column. Most of this ice will float to the surface but some will stay in suspension.

The best case situation for ice at the tailrace would be that nucleation does not occur until the bulk of the flow has passed the downstream end of the eddy. The worst case outcome would be if ice forms on the surface of the eddy and thickens in situ over the winter. The daily water level fluctuations would prevent the ice from adhering to the bank (riprap along the toe of the North Spur) and the ice cover would simply move up and down in response to changing water levels. Assuming 12 hours of constant water levels, the thickness of the ice that attaches itself to the

[4] At turbine efficiencies of 90 percent, about 0.030 MW of heat per m^3 of discharge would be generated at a head of 30 m. This would increase water temperatures by about 0.007 ^{0}C.

shore line will be at most 0.06 to 0.08 m. It should fail in flexure relatively close to the shore as water levels fluctuate, so that not much surface ice would form against the bank.

In the worst case situation, frazil will form in the area next to the eddy and be entrained into the eddy and accumulate there. Since the velocity in the eddy would be relatively low, the frazil will rise to the surface and form ice floes which may or may not congeal to form a stationary cover. After that, it is likely that additional frazil will be entrained into the eddy and accumulate under the stationery ice forming a rather thick deposit of slush within the eddy.

The one factor that could mitigate against any of the outcomes described above is the two dimensional nature of the flow and the rapid water level changes that will develop in response to fluctuating powerhouse flows. Since water level fluctuations are expected to occur on a daily basis, it is unlikely that huge volumes of surface ice would be produced within the eddy and it is unlikely that the surface ice would congeal to a significant extent. Furthermore, the eddy would act somewhat like a lateral storage unit. It would rise and fall as water from the tailrace flows is either injected into or extracted from the eddy. If the outflow from the eddy is rapid enough it is likely that any surface ice that forms within the eddy would be flushed out into the main flow, be transported downstream, and ultimately add to the ice that would be accumulating as the head of the advancing ice cover downstream of the tailrace.

Regardless of which of the above processes actually occur, with about three metres of water level fluctuations at the tailrace, large extents of the riprap will be wetted on a daily basis. As water levels drop, water that has attached itself to the rock/ice will freeze when it is exposed to subzero air temperatures and form a thin film of ice on the wetted portion of the riprap. Each day, as the rock gets wetted and then exposed to cold air, another thin layer of ice will increase the thickness of the ice "deposit" on the rock. The thickness of the ice deposit may eventually be great enough that when the ice deposit becomes submerged, it has enough buoyancy to pluck the rock from the face of the riprapped slope.

Based on ice and rock densities of 900 and 2600 kg/m^3 respectively, a first order analysis indicates that the thickness of the ice deposit would need to be 15 to 20 times the rock diameter to pluck a rock from out of the riprap field. For rock diameters between 300 and 500 mm, a thickness of the required ice thickness would be between 4.5 and 7.5 m – a huge and unattainable thickness. Note that the upper limit of that could develop each day would be at most 0.02 m. Between 225 and 375 cycles or between 7.5 to 12.5 months of operation would be necessary to attain that much ice – clearly much longer than the length of the worst winter in record.

DESIGN ICE CHARACTERISTICS

Dynamic, wind-driven ice forces can develop during both the freeze-up and breakup periods. From the perspective of the ice characteristics, the calculation of these ice forces is based on considerations of the ice thickness, ice strength, and modulus of elasticity.

At freeze-up, when the movement of ice "occurs at temperatures considerably below its melting point" the S6 code [5] suggests that an effective ice strength (crushing strength) of 1500 kPa be adopted. This crushing strength has a significant temperature-related component that will not necessarily translate into a correspondingly high flexural strength. The literature indicates that for cold conditions, the upper limit of the flexural strength is less dependent on temperature-related effects and is affected more by crystal size. The corresponding flexural strength would be about 1000 kPa. The corresponding modulus of elasticity would be in the range of 1000 to 5000 MPa.

During the spring period, ice strength is significantly lower than the mid-winter ice strength, but it is more difficult to define explicitly because it is difficult to determine the date of incipient instability. Therefore, for evaluating the ice forces on structures, it would not be unrealistic to adopt a somewhat conservative ice strength of 700 kPa (corresponding to "ice that breaks up at melting temperatures and is somewhat disintegrated" as defined in the S6 code) because it is likely that ice with that strength could be encountered within an area of an ice floe that could interact with a relatively small structure. The corresponding flexural strength would be 600 kPa and the modulus of elasticity would be between 900 and 3000 MPa.

For the situation where large ice sheets interact with a shoreline over much larger scales (say the diameter of the floe), it would be appropriate to adopt a lower effective ice strength, say 400 kPa, that defines the strength of ice when it is at "a melting temperature and substantially deteriorated". The corresponding flexural strength would be 350 kPa and the modulus of elasticity would be between 500 and 1000 MPa.

Table 4 summarizes the adopted design ice characteristics for freeze-up and breakup ice force events.

Table 4 Adopted design ice characteristics

Situation	Ice Thickness (m)	Crushing Strength (kPa)	Flexural Strength (kPa)	Modulus of Elasticity (MPa)
Dynamic ice forces during freeze-up period	0.20	1500	1000	3000
Uplift forces in late winter	1.2	1500	1000	3000
Dynamic ice forces during breakup – local interactions	0.80	700	600	1500
Dynamic ice forces during breakup – broad-scale interactions	0.80	400	350	1000

[5] Canadian Standards Association, 2000. Canadian Highway Bridge Design Code – CAN/CSA S6. National Standard of Canada, CSA International, Toronto, Ontario.

ICE FORCES ON ROCK-FACED SLOPES

There are three major types of ice action that can destroy the integrity of a riprap placed on a side slope.

1. Plucking of individual rocks from the side slope by rising ice levels when the rocks are imbedded in the ice sheet.
2. The displacement of large areas of rock (bulldozing) by wind driven ice sheets where the stationary ice sheet transmits the force of the wind against the slope.
3. The displacement of individual rocks or small sections of the riprap by large moving ice floes as they strike the slope.

The calculation of these ice forces and their subsequent effects on the integrity of the riprap is based on the following parameters.

1. Ice thickness, flexural strength, and modulus of elasticity.
2. Wind speed, direction, and fetch.
3. Rock size, thickness of riprap layer, and side slope.

The ultimate objective is to define the driving forces on the basis of the ice characteristics and the wind conditions, and to determine the size of rock, thickness of riprap layer, and the side slope that are required to withstand the adopted design ice force.

Ice Plucking due to Water Level Fluctuations

This process is similar to that which produces uplift forces on bridge piers. Ice adheres to the individual rocks and the ice lifts the mass of rocks to which it has adhered when the water level increases. The uplift force is based on the maximum vertical force that can be applied to the edge of the sheet without it breaking in flexure, assuming that the ice sheet can be treated as a beam on an elastic foundation. Since this process can occur anytime during the winter, it is not unreasonable to assume a high flexural strength of 1000 kPa (***Table 4***) because the ice would have no internal deterioration.

The current rule of thumb is that the diameter of the rock should be close to the thickness of the ice sheet to prevent plucking. This somewhat conservative approximation suggests that the minimum diameter of rock to ensure stability should be in the order of 1000 mm if applied to northern areas. In fact, substantially smaller rock sizes appear to function very well under these relatively severe ice conditions.

An alternative view is that the mass of rock that resists the upward movement of the ice sheet is related to the depth that the water penetrates into the rock layer (and subsequently freezes), and corresponds to the volume of rock that is embedded in the ice plus the rock on the slope directly above where the ice level intersects the impervious slope under the riprap. In this situation, the rock mass can be calculated from considerations of the porosity and thickness of the riprap, the slope angle of the embankment, and the adopted design ice thickness. In reality, the plucking resistance is related to the thickness of the riprap layer rather than the actual size of the rock. Note, however, that larger rock will result in thicker riprap layers and subsequently it will be able to resist larger uplift forces.

For the winter ice characteristics summarized in ***Table 4***, a side slope of 3H:1V, and an efficiency factor of 0.75 (the fraction of theoretical rock volume that actually resists the ice movement), the required riprap thickness is 460 mm to prevent disturbance of the riprap. The choice of the efficiency factor is somewhat subjective, but it is meant to reflect the imperfect analysis of the ice-rock interaction and to provide somewhat of a safety factor to the calculation. ***Table 5*** summarizes the required thickness of rock to prevent wholesale rock plucking for the three structures. It is clear that the ice plucking is a relatively minor issue.

Table 5 Thickness of riprap required to minimize rock plucking

Location	Side Slope (xH:1V)	Required Thickness of Riprap to Prevent Plucking (mm)	Actual Thickness of Riprap (mm)
South Dam	1.7	720	900
Upstream shore of North Spur	3.1	450	1500
Upstream shore of North Spur	4.0	360	1500
Downstream shore of North Spur	3.0	460	1800

Ice Force from Stationary Ice Sheet

This situation occurs when the ice sheet is intact and more or less frozen to the shore so that it cannot move without displacing shore material or sliding up the shore, and it is free to transmit the entire wind and current shear to the downwind shoreline over a significant width. While it is clear that the irregular shoreline of the reservoir will prevent the transmission of the full force to the embankment, a reasonable design assumption is that the force per unit length at any location along the embankment is derived from the accumulated shear per unit width of the ice sheet over its fetch. ***Table 6*** summarizes the force per unit width on each of the embankments on the basis of the design wind speeds in ***Table 3*** and a nominal current velocity of 0.10 m/s. Obviously, the unit force scales with the fetch length.

Table 6 Ice forces from a stationary ice sheet

Structure	Adopted Fetch Perpendicular to Face of Structure (m)	Ice Force Per Unit Width (kN/m) [4]
South Dam	1200 [1]	1.6
Upstream shore of North Spur	2500 [2]	5.0
Downstream shore of North Spur	1200 [3]	0.8

(1) Distance between face of South Dam and the ice boom.
(2) Distance between shore and intersection point of a line perpendicular to the shore and a line along the centreline of the upper reservoir.
(3) Distance between the shore and east boundary of largest potential ice accumulation zone in the large eddy along the north bank downstream of the tailrace.
(4) Based on a nominal drag coefficient of 0.002 between smooth ice and air [6].

The ice/riprap interaction can be quite variable, depends on the configuration of the contact area between the ice and the riprap, and is difficult to quantify explicitly. At one end of the spectrum, the ice rides over the riprap and the forces on the individual rocks are based on the friction coefficient between the moving ice and the stationary riprap. At the other end of the spectrum the ice bulldozes its way into the riprap and displaces the rock up the slope. In this situation, the force required to move the rock depends on the mass of the rock that is being affected by the impinging ice sheet and coefficient of friction between the rock and the earth slope. In both cases, the magnitude of the force depends on the slope of the structure and may be limited by either the fetch-limited wind drag or the maximum horizontal force that the ice sheet can sustain without breaking in flexure – again assuming the sheet acts as a stiff plate on an elastic foundation.

For the benign forces identified in ***Table 6***, considering only the force required to push the ice up the slope (the ice sheet is already fractured) and assuming a very conservative friction coefficient of 0.10, the ice would ride up the slope to a height of at most a few tenths of a metre. Alternatively, if the ice was to bulldoze into the riprap over a generally wide area, the ice force would be insufficient to move the rock up the slope. At worst, the force would be transmitted through the rock into the embankment and produce a deflection within the granular of something in the order of centimetres assuming even the most conservative spring constant. In this situation, the fetch is too short to produce enough drag on a large sheet abutting against the riprap to generate a significant force on the riprap.

[6] Prinsenberg, S. and I. K. Peterson, 2002. Variations In Air-ice Drag Coefficient Due to Ice Surface Roughness. International Journal of Offshore and Polar Engineering, Volume 12, Number 2.

Ice Forces from Moving Ice Sheet

Ice forces from a moving ice sheet would likely be the most severe when large ice floes that are mobilized by the wind are driven into the embankment during the melt period when significant open water has been created along the shoreline. The ice force is related to the size of the ice floe, its velocity, and the way in which it interacts with the embankment. The floe sizes could be quite variable, but an upper limit of floe diameter of 300 m would not be unreasonable in the reservoir and at most 100 m in the area upstream of the South Dam. The corresponding floe velocity would be about 0.8 m/s and 0.6 m/s for the two locations if it assumed that the floes reach their terminal velocity – a somewhat conservative assumption – based on a drag coefficient of 0.002 between smooth ice and air.

The nature of the interaction and the type of ice failure is the most important consideration in determining the magnitude of the ice force. Any number of scenarios could occur but the following are the three most likely – (i) crushing of the ice sheet against the face of an individual rock and displacing that one rock, (ii) sliding of the floe over the top of the riprap without mobilizing the riprap, and (iii) crushing of the ice floe against a number of rocks so that the ice force is distributed in a limited way along the embankment.

Movement of Individual Rocks

Forces on an individual rock would depend on the crushing strength of the ice sheet. However, given the kinetic energy of the large floes and the minimal contact area that could exist (in the order of the diameter of the rock) it is likely that the full crushing force could develop on an individual rock, regardless of its size and the crushing strength of the ice. For a typical arrangement of rock within the riprap, the ice force would tend to rotate the rock downward into the embankment as the ice crushes through the rock. This prevents the individual rocks from being rotated out of place – instead they are bulldozed into the side slope while driving the rock located above the ice level further up the slope. So, while some of the rocks would be displaced locally, the riprap would likely remain intact. In fact, observations in the laboratory indicate that once the ice melts the displaced rock often falls back into the area from which it was displaced. Inspection of the riprap and occasional maintenance is the most reasonable way to deal with this type of disturbance.

Bulldozing

With respect to bulldozing, there are no well established procedures that properly relate the stability of riprap to the kinetic energy of large ice floes. Furthermore, there are no criteria to define what might be an acceptable volume of rock movement should the ice floes have enough energy to displace the rock. One criterion might be to limit the horizontal penetration of the ice into the riprap to less than that required to expose the slope/filter fabric. In this situation, the distance that the ice floe can penetrate into the riprap depends on its initial kinetic energy and the amount of energy required to move up-slope the volume of rock that would be displaced during penetration by the ice floe. Since the distance to the embankment though the riprap is a function of both the thickness of the riprap and the slope angle, a shallower side slope and a thicker riprap layer will provide greater protection against the exposure of the side slope.

Notwithstanding the limitations that would arise from the considerations of the kinetic energy of the floes, Sodhi [7] carried out a set of systematic model studies of ice/riprap interactions using reasonable scaled riprap gradations with urea-doped ice to properly scale its strength and stiffness. This work provides an assessment of the effects of a worst-case ice strength limited (not energy limited) condition on the local stability of riprap. Even so, his results are somewhat equivocal because of somewhat inconsistent indications of damage over a wide range of both ice forces and rock sizes.

Fundamentally, the worst-case ice strength assumption inherent in the analysis is reasonable. It would be unlikely that the ice sheet would fail if the ice/rock interaction occurred over a wide area simultaneously because the drag forces on the ice sheet would be too low given the limited fetch (see earlier discussion). However, if the interaction occurred over the width of one or two rock diameters there would be more than enough horizontal force due to the drag on the ice sheet to cause a local bending failure in the ice sheet. The upper limit of the vertical force to cause a failure in the ice sheet would then limit the horizontal force, which in turn would limit the upslope force that would act to disturb the riprap.

If the horizontal ice force on an individual rock acting over a width that is characterized by the D_{100} of the size distribution of the rock is resolved into its upslope component, and that force is compared to the down-slope component of the weight of an individual rock of size D_{100}, the riprap is either totally stable or experiences some minor instability when the resisting force is greater than about 60 to 200 times the ice force, according to Sodhi's data (***Figure 7)***. Since the maximum upslope force is related to the thickness and the flexural strength of the ice sheet, it is possible to define a range of more or less stable rock sizes for the adopted embankment slopes and the adopted design ice characteristics.

It should be noted that Carter looked at the Sodhi data in a similar way. However, he assumed that the ice fails in crushing. This is a more conservative approach that results in significantly larger rock sizes for the same ice conditions than the approach adopted herein. For typical side slopes used in most structures, the ice almost always fails in bending due to its low flexural strength. It is not clear which approach best represents the actual failure mode in the field. It is clear, however, that the ice tended to fail in bending in the Sodhi model.

Table 7 summarizes the worst case ice forces that could develop for a range of side slopes, ice thickness, and ice strengths; and the corresponding rock size that would be stable under those conditions. The suggested minimum D_{50} of riprap to prevent its extensive displacement due to ice forces arising out large floes striking the riprap varies slightly with the side slope and the adopted flexural strength, but it would be between 200 and 290 mm – typically Class I riprap.

[7] Sodhi, D., S. Borland, and J. Stanley, 1996. Ice Action on Riprap – Small-Scale Tests. CRREL Report # 97-17. U.S. Army Corps of Engineers, Hanover, New Hampshire.

Table 7 Maximum possible ice forces and corresponding stable rock sizes, ice thickness = 0.80 m and modulus of elasticity = 1500 MPa

<table>
<tr><th rowspan="3">Side Slope (xH:1V)</th><th rowspan="3">Flexural Strength (kPa)</th><th colspan="3">Maximum Ice Force (kN/m)</th><th colspan="2">Stable Rock Diameter (mm)</th></tr>
<tr><th rowspan="2">Vertical</th><th rowspan="2">Hori - zontal</th><th rowspan="2">Up Slope</th><th colspan="2">R/F = 200</th></tr>
<tr><th>D_{100}</th><th>D_{50}</th></tr>
<tr><td rowspan="2">1.7</td><td>600</td><td>15.6</td><td>14.0</td><td>12.0</td><td>300</td><td>200</td></tr>
<tr><td>1500</td><td>26.0</td><td>23.3</td><td>20.0</td><td>400</td><td>270</td></tr>
<tr><td rowspan="2">3.1</td><td>600</td><td>15.6</td><td>8.7</td><td>8.3</td><td>330</td><td>220</td></tr>
<tr><td>1500</td><td>26.0</td><td>14.4</td><td>13.8</td><td>420</td><td>280</td></tr>
<tr><td rowspan="2">4.0</td><td>600</td><td>15.6</td><td>7.4</td><td>7.2</td><td>340</td><td>225</td></tr>
<tr><td>1500</td><td>26.0</td><td>12.3</td><td>11.9</td><td>440</td><td>290</td></tr>
</table>

Large ice floes could also interact with the embankments during the freeze-up period. For a typical design thickness of 0.20 m, a flexural strength of 1000 kPa, and a modulus of elasticity of 3000 MPa, the maximum up-slope force would vary between 2.6 and 4.5 kN/m for the range of side slopes in ***Table 7***. The D_{50} of the stable rock would be about 150 mm. Clearly, the thicker ice during the melt period results in a more severe ice forces even though it has a lower modulus of elasticity.

Up-Slope Sliding of a Large Mobile Ice Floe

Forces due to the sliding of the ice sheet over the riprap would develop independent of the rock sizes in the riprap since the friction force along the face of the riprap would be quite low. The ice would slide up the face of the riprap until all the kinetic energy of the ice floe was converted to potential energy. The height that the ice would achieve would depend on the work required to move the ice floe up the slope. In this situation, it is not the integrity of the riprap that would be the issue, but the amount of freeboard provided between the initial ice level and any structures on top of the embankment.

Assuming that the energy required to break the ice in bending is negligible (a conservative assumption), the maximum height that the ice can be pushed up the slope depends on the kinetic energy of the ice floe (a function of its mass and velocity), the slope angle, the friction coefficient between the ice and the riprap, the ice thickness, and the width of the ice being pushed up the slope (some fraction of the diameter of the ice floe). A first order analysis suggests that that ice will slide up the slope at most about 0.5 to 1.5 m before the energy of the floe is used up by both friction and changes in potential energy – again this does not appear to be a significant problem.

LOG/ICE BOOM

Boom Characteristics

It is intended that the ice boom will be located about 1100 m upstream of the spillway and powerhouse in water that would be about 17 m deep on average with a maximum depth of about 31 m. The preliminary plans call for the boom to extend from the west flank of Innu Spirit Mountain southwest to the south shore of the reservoir. The boom would have a bearing of about 230 ^{0} from north and the wind direction perpendicular to the bon chord would be 320 ^{0} from north. The length of the boom chord will be about 600 m plus or minus, but the boom itself will be somewhat longer owing to its parabolic shape. The boom will likely be composed of spans that would be no longer than 100 m with the end of each span anchored to the bed of the reservoir with anchor cable.

It is intended to install the boom in the autumn when the reservoir level is at an elevation of 25 m. The flow though the reservoir will depend on how the facility is operated during the transition period between when all the inflow is passed through the facility and when the reservoir levels start to be raised. Should an ice cover form while the reservoir is being maintained at an elevation of 25 m the flow through the reservoir would be between 1000 and 2000 m^3/s – the expected flow would around 1800 m^3/s. At the expected flow of 1800 m^3/s, the approach velocity and Froude number would be about 0.50 m/s and 0.050 respectively. This is just on the edge of the ability for a boom to operate and it may be difficult for the boom to prevent some of the larger ice floes from being entrained. Assuming that reservoir levels were in the process of being raised, the flow through the reservoir would be about 540 m^3/s. The approach velocity and Froude number would be about 0.13 m/s and 0.010 respectively. For this condition there should be no problem getting the ice to accumulate against the boom.

During post-project operations with the reservoir at elevation 39 m, the mean flow velocity at this point will be in the order of 0.24 m/s with the flow Froude number of about 0.018 for a likely nominal worst-case reservoir flow of 2600 m^3/s (***Figure 8***). Under quiescent wind conditions the floating ice will accumulate against the boom and the leading edge of the surface ice accumulation will advance upstream from the boom as ice arrives at the leading edge. Strong and frequent winds are expected from the northwest and they will drive the ice up against the boom during the freeze-up period when there would be ice floes on the reservoir and during the breakup period when the ice has deteriorated to the point where it could be mobilized by the wind. Ultimately, the magnitude of the force or the so called line load on the boom will depend on how much ice accumulates against the boom and the overall shape of the accumulation.

Boom Design

Obviously, the intention of the boom is to protect the powerhouse and spillway from the effects of wind and current driven ice and debris. The boom should be as short as possible, but positioned where it can most effectively shield the facility. Furthermore, the boom should be rugged and simple to install and maintain. Booms typically have a maximum operational span width of about 100 to 130 m ***to limit the boom deformation when the wind direction is skewed relative to the boom span***, and a maximum boom length of about 10 to 15 percent greater than the span width.

Two stability criteria must be met for the boom to operate effectively. First, the force on the individual boom pontoons must be less than the force required to submerge the pontoon. If the force is greater than the submergence force, the pontoon will be overtopped and debris will flow over the boom. Thus, the shape, size, and buoyancy of the pontoon can be specified on the basis of the expected force and whatever submergence criterion is adopted.

The second stability criterion relates to the integrity of the boom cable. The boom cable should be of sufficient strength to withstand the tension that is generated by the submergence forces on the pontoons acting along the entire length of the boom. Thus, the submergence criterion not only governs the design of the pontoons, but also the boom cables, connectors, and the boom anchor points. In many boom designs, the limiting condition is the stability of the pontoons. It is preferable that, should a catastrophic load develop, the boom fails by overtopping instead of by breaking of the boom cable(s).

Line Forces and Forces on Cables

Currents, winds, and waves exert forces on the accumulation of ice or debris upwind of the boom and these forces are transferred to the boom. Typically wave forces are small and are not a serious concern. The force per unit area of the accumulation that is produced from the consideration of wind and the currents is integrated over the area of the accumulation. The area of the accumulation that develops forces on the boom depends on its shape. Typically, the accumulation is in the shape of an isosceles triangle with the base of the triangle defined by the span of the boom and the height of the triangle (length of the accumulation) defined by its apex angle. The forces that develop from the portion of the accumulation that is outside the isosceles triangle are transferred to the banks or the shore and have little impact on the boom.

Apex Angle and Surface Area of Accumulation

The apex angle depends on the angle of internal friction of the material in the accumulation. The internal friction is reasonably well defined for ice although it has a large range, depending on geometric circumstances. Analysis of river ice suggests that the apex angle of the accumulation should be about 15 to 20 ^{0}, yet observations of ice accumulations behind booms on lakes and in the St. Lawrence River indicate that the apex angle can be as large as 90 ^{0}. ***Table 8*** summarizes the area of the ice accumulation for a conservative range of apex angles that could affect a boom with a length of 600 m.

Table 8 **Surface area of accumulation and expected unit line forces for various apex angles of an ice accumulation against a 600 m long boom**

Apex Angle (^{0})	Length of Accumulation (m)	Area of Accumulation (m^2)	Unit Current Force (kN/m)	Unit Wind Force (kN/m)	Total Unit Line Force (kN/m)
10	3430	1,030,000	0.033	3.03	3.06
20	1700	510,500	0.025	1.50	1.53
30	1120	336,000	0.020	0.99	1.19
40	825	247,500	0.017	0.73	0.75

Current and Wind Forces

Figure 8 shows the longitudinal variation of the average velocity and the Froude number in the area where the boom is to be located. At km 43.7, the velocity and Froude number are quite high – 0.37 m/s and 0.029 respectively at the worst case flow of about 2600 m^3/s – and the local hydraulic conditions would be more severe by a significant margin than just a few 100s of metres upstream. It is suggested that the boom be placed approximately at or upstream of km 43.8 to reduce the effects of the high velocity zone on its performance. Clearly, there will be a need to chose the boom location on the basis of an optimized length and level of expected performance.

At the FSL, the maximum and mean flow depth immediately under the boom will be about 31 and 17 m respectively. The average depth under the ice accumulation upstream of the boom will be about 20 m and the average velocity will be about 0.15 m/s, but decreasing to 0.090 m/s at the upstream end of the accumulation. The total force generated by combined winds and currents will depend on the adopted value of the apex angle. For the worst case apex angle of 10 ^{0}, and assuming a typical water drag coefficient of about 0.0020, the average current shear force would be about 0.033 N/m^2. For a design wind of 20 m/s and a wind drag coefficient of 0.0034 at the surface of an accumulation of broken ice, the design wind shear on the ice accumulation would be 3.03 N/m^2. ***Table 8*** also summarizes the unit line forces for a 600 m long boom for various apex angles.

It is evident that the forces and the subsequent line loads are quite small compared to the line loads on booms in other environments. For example, ice booms in rivers are usually designed to withstand a line load of up to 10 kN/m. Even for the most conservative apex angles the forces on the boom are lower than that.

Forces on Cables

The shape of the boom under its fully loaded condition would be parabolic. However, the characteristics of the parabola depend on the direction that the load is applied. For example, if the load is perpendicular to the boom span, the parabola will be symmetrical within the two end points on the boom and the tension in the anchor cables will be equal and at a maximum at each of the end points. ***Table 9*** summarizes the tensions in the cables at the two end points for different apex angles assuming that the cable length is 6 percent (sag ratio of 0.15) greater than the span length. The maximum tension in the span cables would vary between about 75 and 300 kN – forces that steel cables of between 15 and 25 mm in diameter would accommodate in the absence of any safety factor.

Table 9 Cable tensions for a 100m span length with a 106 m cable length (sag ratio = 0.15) for a range of apex angles

Apex Angle (^{0})	Line Load (kN/m)	Total Force (kN)	Maximum Cable Tension (kN)	Minimum Cable Tension (kN)
10	3.06	306	300	255
20	1.53	153	150	125
30	1.19	119	115	100
40	0.75	75	75	65

If the load is not perpendicular to the boom span, the parabola will not be symmetrical and the tension in the cable will be greater at one end of the span than at the other end. Allowances for skewed accumulations need to be made in the design of the cables. Nevertheless. it is evident that a boom would be feasible from a layout and cabling perspective. However, the final configuration (boom length, span lengths, optimal sag length, and arrangements of anchors and cabling) needs to be optimized.

Pontoon Characteristics

A number of types of pontoons could be used to catch the ice and debris. Square or rectangular wooden pontoons are often used, but these eventually become water logged and lose their buoyancy. Furthermore, they deteriorate from debris impacts and from impacts between the individual pontoons. Round steel pontoons are also used in many applications. They are simple to construct by welding caps on the ends of steel pipe, they are rugged, and they provide considerable buoyancy (relative to wooden pontoons) as long at they do not leak. The steel pontoons could be filled with styrofoam to prevent loss of buoyancy should the skin of the pontoon rupture. Furthermore, it is simple to weld to the steel pontoons the chains or cable attachments that connect the pontoons to the span cable. High density polyethylene (HDPE) piping has been used in some very benign conditions, but the material would be too fragile to use in this environment. The choice of the wall thickness needs to be optimized to provide sufficient stiffness to the pontoon without reducing its buoyancy. Nominal wall thickness values would be chosen on the basis of maximizing buoyancy, and the structural stiffness of the pontoon with the selected wall thickness would need to be checked in the final design.

The required pipe diameter to ensure that pontoons are not submerged and overtopped can be determined by balancing the submergence force (net buoyancy) of the pontoon against the vertical component of the line load (described as the line load times the coefficient of friction between the debris and the pipe) and the location where it acts on the pontoon. The coefficient of friction is somewhat difficult to describe because it reflects the outcome a large number of types of interactions between the ice/debris and the pontoon. During the freeze-up period ice will attach itself to the pontoons, thereby making them more buoyant and increasing the submergence force. While the stability analysis has not been undertaken herein, experience suggests that relatively small-diameter steel pontoons could withstand the expected line loads summarized in ***Table 8***.

Table 9 summarizes the capacity of steel pontoons of various diameters, assuming conventional arrangements of anchor cables, span cables, chain linkages, and junctions plates as calculated by Abdelnour [8] in his work on the Lake Erie –Niagara River ice boom. It is evident that the expected line loads for the design can be sustained by pontoons with diameters in the range of 600 to 750 mm. However, it is clear that more design work will be required to define the optimal pontoon type and diameter.

Table 9 Ice overtopping line loads for steel pontoons of varying diameters

Pontoon Diameter (mm)	Overtopping Load (kN/m)		
	Adverse Conditions	Average Conditions	Optimal Conditions
500	0.5	1.0	1.5
600	2.0	2.7	3.7
750	4.0	6.0	8.0

Boom Installation

One of the issues that pertains to the installation of the ice boom is its timing. It would be far easier to install the boom prior to the reservoir being filled since the bulk of the work could take place in the dry. However, the anchor cable lengths would need to accommodate an increase in water levels of approximately 22 m as the reservoir level increases from about 25 m to its FSL. Furthermore, since the reservoir is expected to be filled at least partly over a winter, the pontoons would become encased in ice and as the ice cover is lifted a tremendous amount of tension could be placed on the anchor cables.

The lengths of the anchor cables are typically much longer than the depth of water in which the boom is installed. For example the Lake Erie ice boom at the entrance to the Niagara River is

[8] Abdelnour, R., R.D. Crissman, and G. Comfort. 1994. Assessment of ice boom technology for application to the Upper Niagara River. In Proceedings of the IAHR Symposium on Ice, 23-26 August, 1994. Trondheim, Norway. Vol 2, pp. 734-743.

located in only about 6 m of water and has anchor cables that are about 160 m in length. The ice boom on the Allegheny River at Lock and Dam #8 is located in 8 m of water and its anchor cables are 75 m long. The angle that the anchor cable makes with respect to the bed for the Allegheny boom is about 6 degrees and for the very long Lake Erie boom the angle is only about 2 degrees. The optimal angle is a parameter that is not discussed too much in the literature, but it is unlikely to be greater than 10 degrees.

Assuming that an angle of 10 degrees is adopted for the Muskrat ice boom, the length of the anchor cables would be about 120 m based on a an assumed water depth of 27 m (average bed level is at El.12 m at the anchor point and FSL is at El. 39 m). Prior to reservoir filling the angle of the anchor cables would be about 3 degrees. Over the 14 m range of water level increase from 25 m to 39 m, the pontoons would want to move upstream about 1.6 m or an average of 0.11 m for every metre in water level rise. Given the storage-elevation curve produced by SNC-Lavalin and average net inflows of 1260 m^3/s (1800 m^3/s less the environmental flow of 540 m^3/s) it would nominally take on average about 18 hours to realize a one metre increase in water level. If it is assumed that the water level increase is continuous, the boom will want to move upstream at a continuous rate of 0.006 m/hr.

The upstream movement of the pontoons would mean that the ice cover would need to be moved upstream by either driving it up the left bank of the reservoir or producing a failure in the ice that would allow the boom to adjust to its preferred position. It is unlikely that the ice sheet could be moved in mass or that the adjustment would be so slow that it could be accommodated by creep in the ice sheet. Ice sheet failure would be the only mechanism by which the boom could move upstream, but the failure mechanism would be very complex and the forces on the boom difficult to estimate.

Adopting a somewhat conservative ice growth rate, and assuming that the water levels remain steady for 12 hours during which the ice sheet could form along a pressure ridge in front of the boom, the expected ice thickness that would need to be broken would be between about 0.05 and 0.08 m. As first order estimate, assuming that the ice sheet fails in buckling across the width of the boom, the unit line load on the boom would be about 10 kN/m. To cause the ice sheet to fail in crushing, the force on the boom would be between 75 and 120 kN/m. Clearly, the ice sheet would fail in buckling before it crushes, but even then it may be difficult for the anchor cables to accommodate that force.

However, since there will be a sag in the anchor cable and the expected upstream displacement of the pontoons will not be large, it is likely that it could be accommodated by a change in the sag angle of the anchor cables. As an added measure, slack could be built in to the anchor cables and the span cables to accommodate the additional tension that could result if changes in the sag characteristics are not sufficient. It would be possible to compute the changes in the sag characteristics and/or the manner by which the slack could be taken up as the boom moves upstream and the tension increases on the anchor and span cables. This concept should be investigated more rigorously.

The stability of the ice sheet on the reservoir also could become an issue. A water change of 14 m would increase the average reservoir width at the lower end of the reservoir from about 1200 m to 1500 m over a period of about 10 days. Each day, the width of open water would increase by about 15 m and on average about 0.05 m of ice would be expected to form in that narrow width. The main concern would be that the current forces and the westerly wind acting on the ice cover would adversely affect the ice boom and or the structure itself by moving the ice sheet downstream to fill the strip of open water before the ice growth could provide sufficient resistance to withstand the wind shear. Assuming an operative typical fetch of 1300 m at the downstream end of the reservoir, the unit force for a 20 m/s wind would be about 1.1 kN/m. This force is substantially less than that required to buckle a 0.05 m thick ice sheet. So, it appears that the daily growth of ice within the open water that forms along the banks should be sufficient to prevent the ice sheet from moving. However, a more rigorous two dimensional analysis should be undertaken to confirm the shape of the ice sheet and the distribution of lateral support that would be provided by the irregular shape of the reservoir.

There are other issues that may arise at the head of the ice sheet as it advances upstream with the increasing length of the reservoir. However, these likely would not affect ice conditions at the lower end of the reservoir. Note, however, to minimize these effects it may be prudent to start filling the reservoir only after a relatively thick ice cover has formed while the reservoir is at El. 25 m.

CONCLUSIONS AND RECOMMENDATIONS

This report provides a review of the ice-related issues that may arise during the filling of the Muskrat Falls reservoir and during subsequent winter operation in the post-regulation period. The scope of the review has been based on a limited amount of hydraulic and meteorologic data and thus should be viewed only as high-level review of the severity of the concerns already identified by the design team. Assessment of the wind/ice/structure interactions has been undertaken based on readily available information and attempts have been made to quantify these processes only so far as to get a first order perspective of the potential effects of the ice conditions. While the analysis and the resulting conclusions may help to identify issues that require more analysis, they should not be used as a basis for design without undertaking a more rigorous analysis within the framework of a conventional design process.

The analysis provided the following conclusions.

1. Ice forces on riprap in the reservoir are not expected to be so severe that they would require rock larger than that required to accommodate the wind/wave effects. However, in the final design/evaluation of the ice forces it is recommended that (i) a consensus be developed on the appropriate design parameters (return period of the design wind, ice characteristics, appropriate safety factors, etc) to be consistent with the current project protocols and (ii) the results of the analysis undertaken herein be confirmed within that context.

2. While it is expected that ice in the tailrace will not adversely affect the riprap along the downstream side of the North Spur, it is difficult to be definitive on this issue on the basis of the data that was examined herein. It is recommended that two dimensional numerical flow modelling be undertaken to assess the behavior of the large eddy along the left bank and to develop a better understanding of the disposition of ice when or if it gets entrained into the eddy. It will also be worthwhile to more rigorously determine the water temperatures in the tailrace and simulate (in two dimensions if possible) the potential generation of both frazil and surface ice for a short distance downstream of the tailrace.
3. It is evident that the local hydraulic and wind environment is suitable for the deployment of a log/ice boom. It should function reasonably well while the reservoir is at full supply level provided the pontoons and the boom cabling are sized appropriately. Nevertheless, the design assumptions made herein should be checked and updated, and a rigorous process followed to prepare a bona fida design of the boom.
4. There is a concern about the deployment of the boom when raising the reservoir during the winter from its construction water level of 25 m to its FSL of 39.0 m. It is likely that the boom pontoons will be encased in ice and it will be difficult to prevent significant tension from developing in the anchor cables. These forces can likely be mitigated by providing additional slack in the boom cables during installation. However, it may prove too difficult to realize this slack in a systematic way as the water levels increase during the filling period. It is suggested that the tension in the anchor cables be modelled numerically to assess how both the sag and the tension would change over the filling period.

Thank you for the opportunity to comment on these ice-related issues and for the assistance and information that you provided during the course of the work. If you have any questions, please give me a call in our Edmonton office at (780) 436-5868.

Sincerely,

northwest hydraulic consultants

David Andres, M. Sc., P. Eng

Figure 1 Monthly air temperatures at Goose A

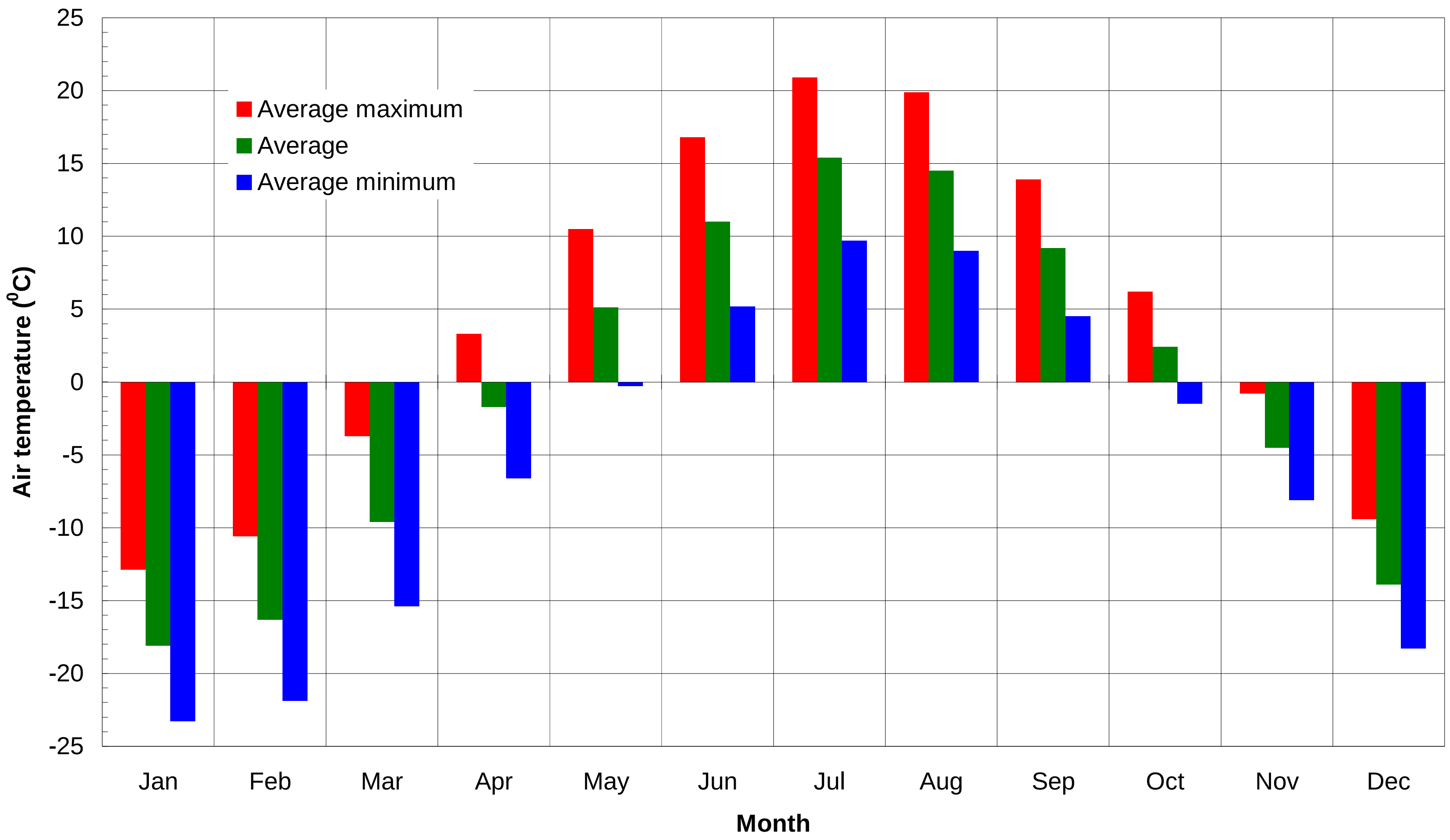

Figure 2 **Monthly snowfall and snow on ground at Goose A**

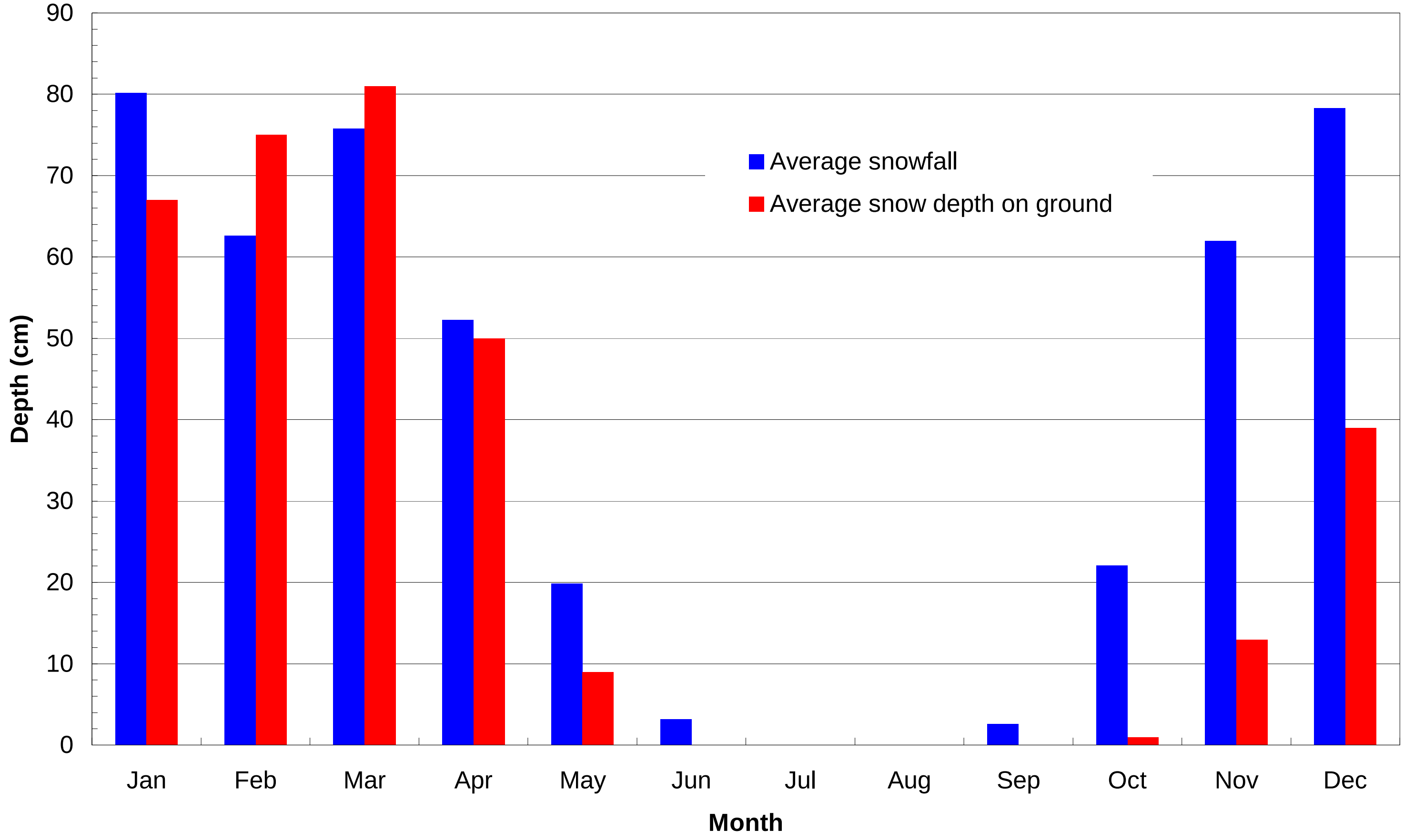

Figure 3 Average solar radiation intensity by month at Goose A (derived on basis of bright sunshine hours)

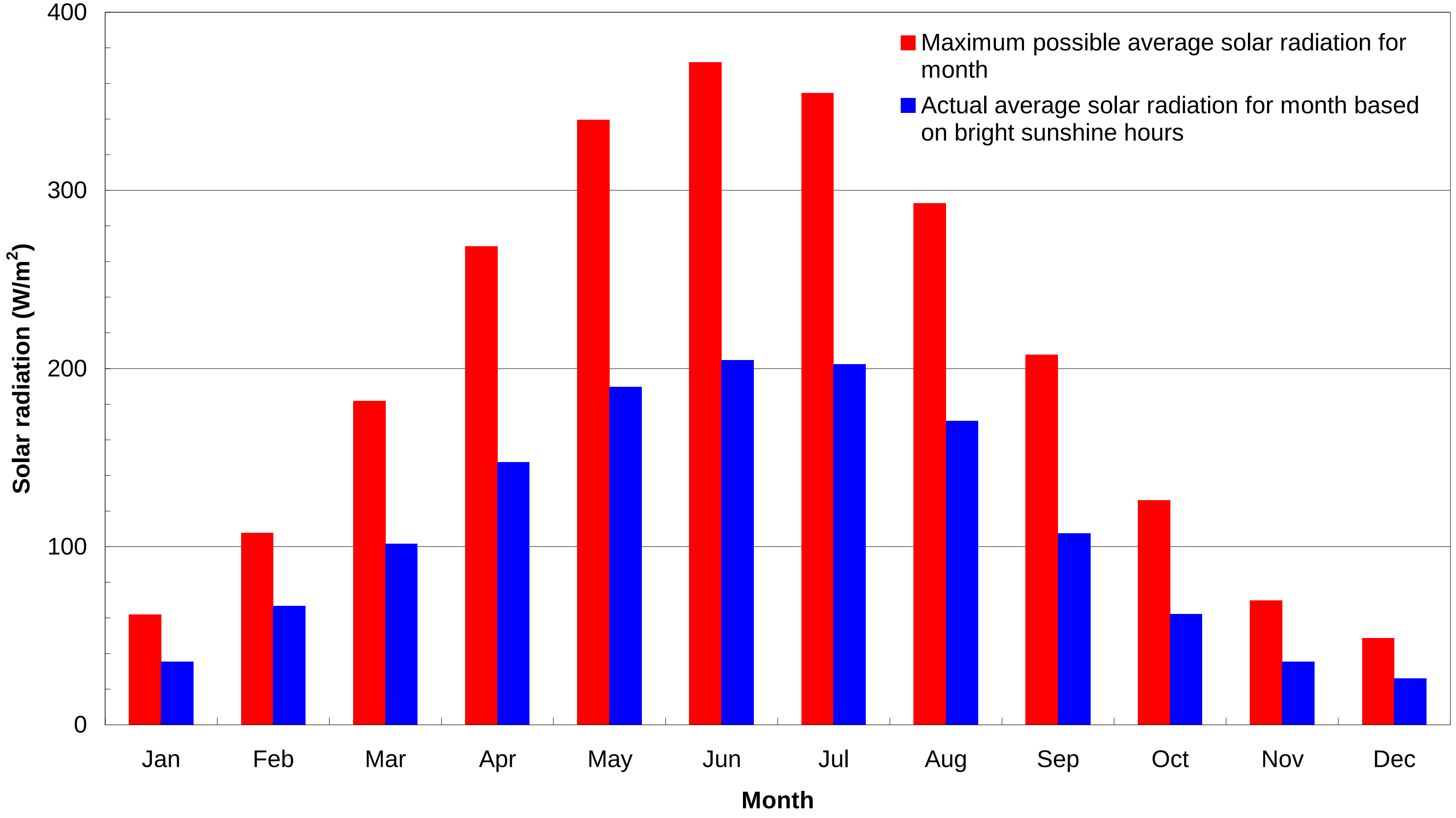

Figure 4 Wind characteristics at Goose A

Fig. 4-1 Percent of time winter wind occurs from a given direction

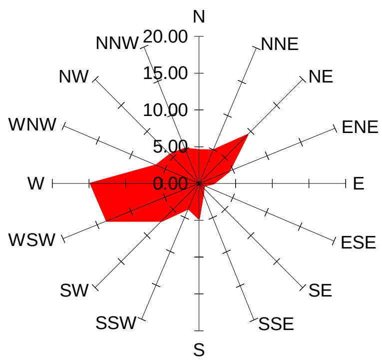

Fig. 4-2 Mean wind speed (m/s) for winter winds from a given direction

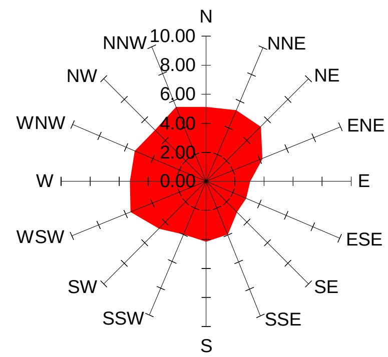

Fig. 4-3 All-direction wind speed statistics

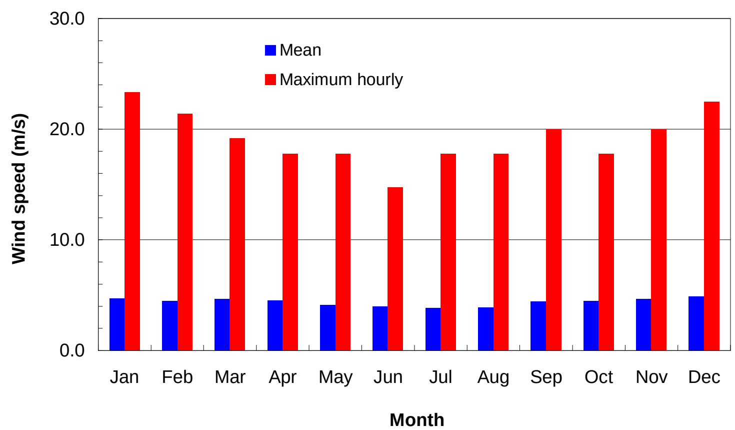

Figure 5 **Mean velocity and Froude number along reservoir**

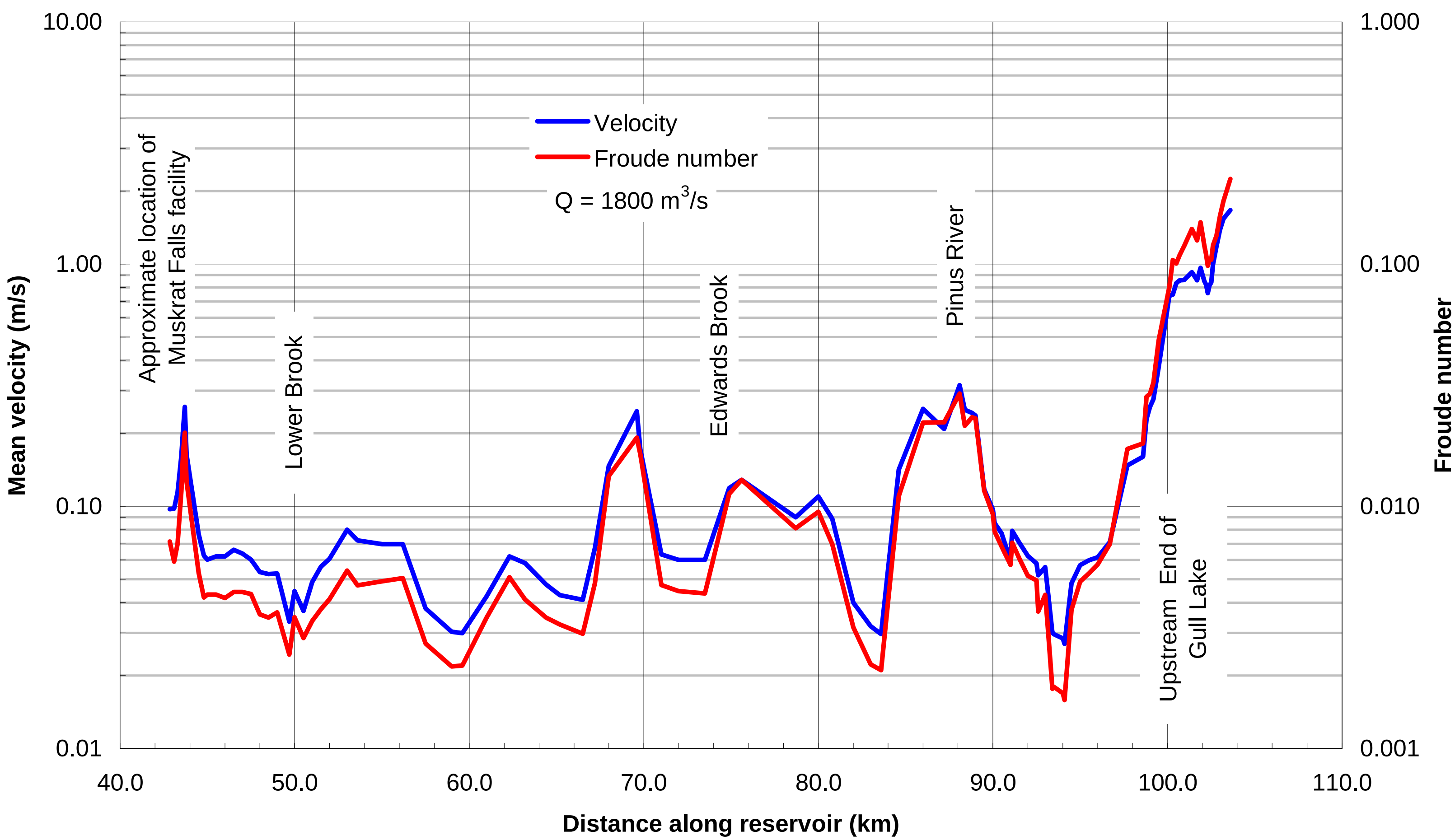

Figure 6 Simulated ice thickness during the spring period (Hatch, 2013)

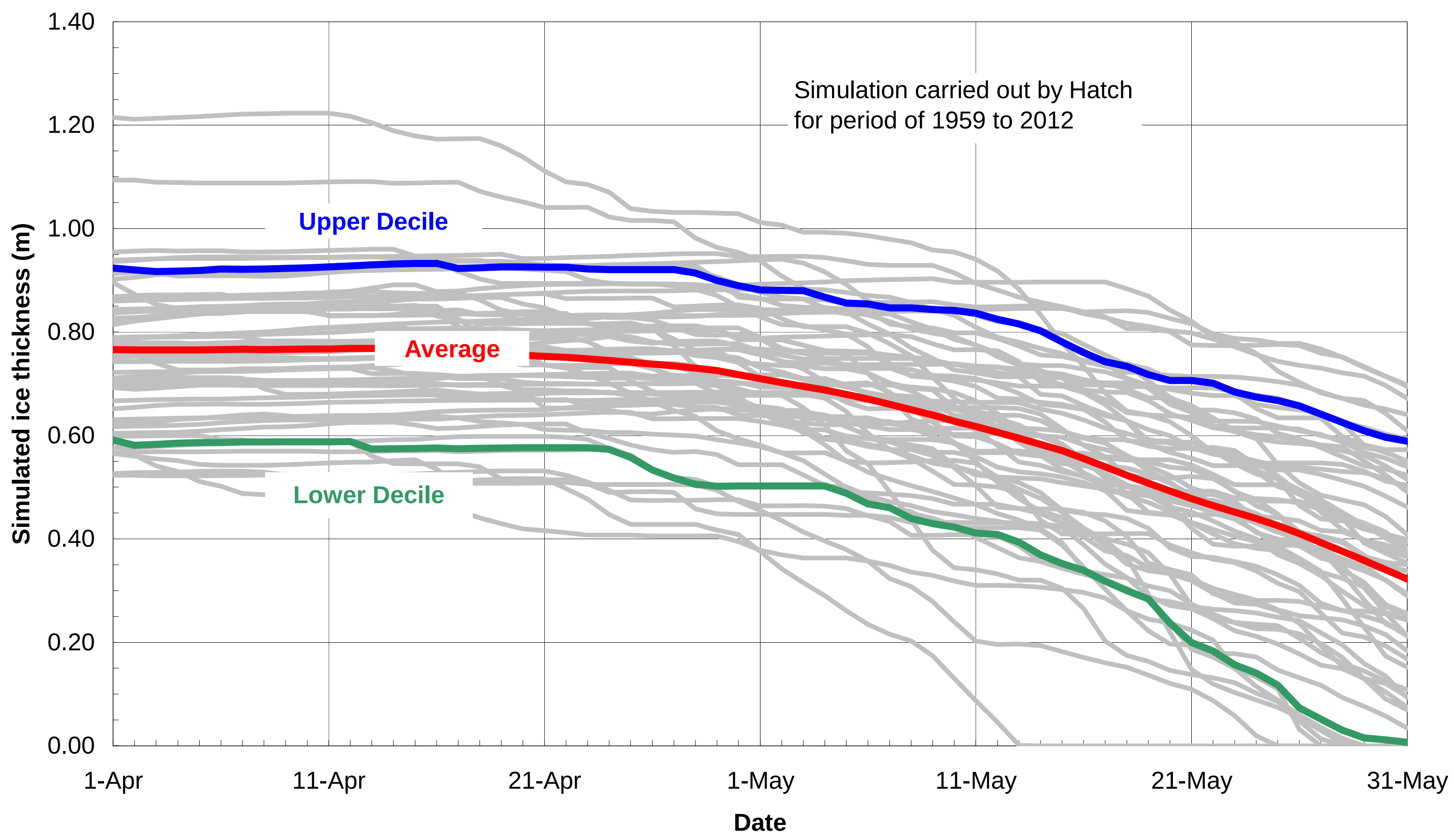

Figure 7 Criteria for stability of rock riprap against ice forces acting on slope

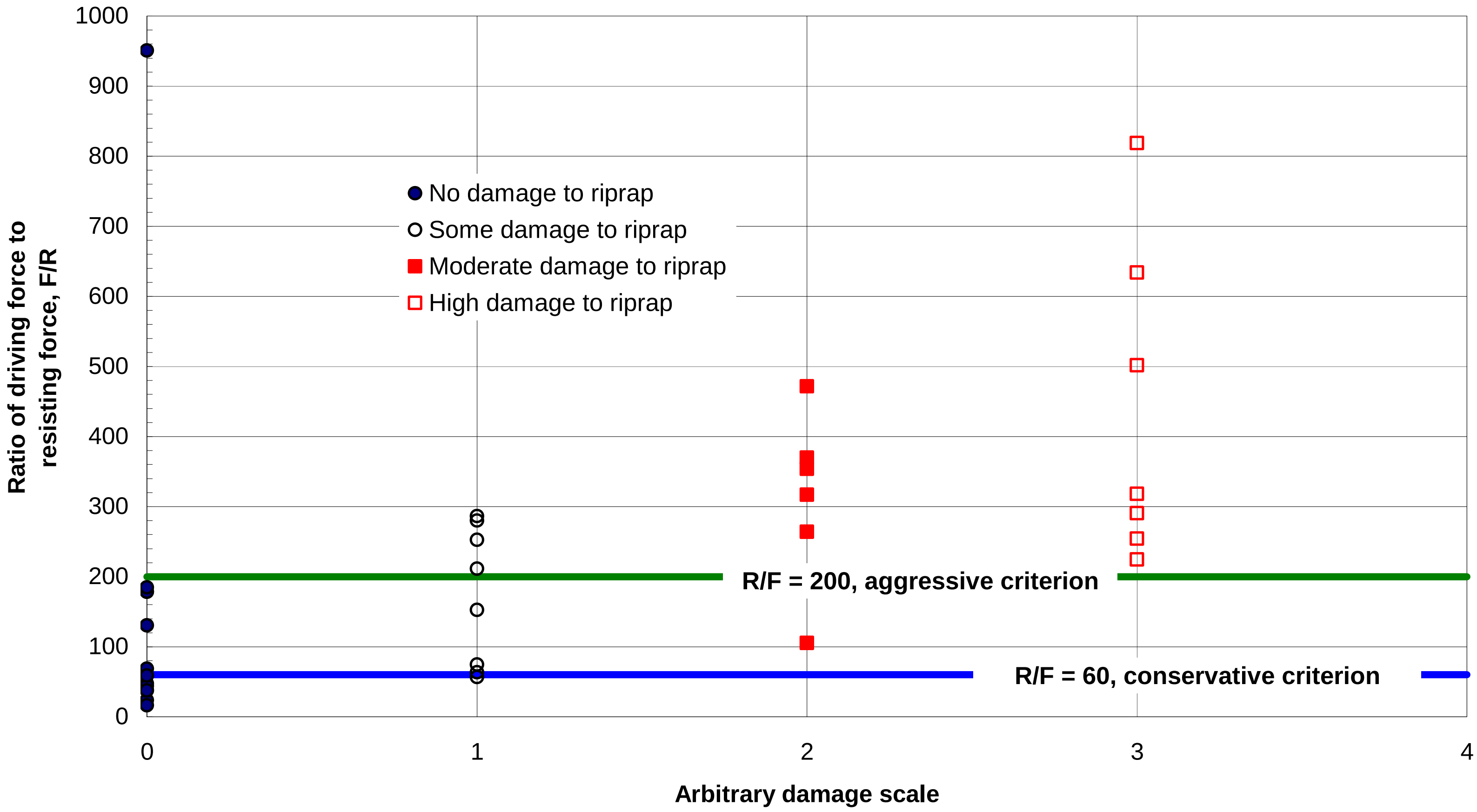

Figure 8 **Longitudinal variation in mean velocity and Froude number for a design discharge of 2600 m^3/s at the location of the ice boom**