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Lower Churchill Project

RIPRAP DESIGN FOR WIND-GENERATED WAVES

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1 Étienne P.D. Mansard et al, 1996, "Model Study of Reservoir Riprap Stability", Chapter 136, Proceeding of 25th ICCE, Orlando, Florida

2 Jean-Pierre Tournier et al, 1996, "An Improved Design Method for the Riprap of Earthfill Dams of Large Reservoirs", Chapter 163, Proceeding of 25th ICCE, Orlando, Florida

3 CERC (US), 1984, "Shore Protection Manual", U.S. Army Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, Virginia, 3 Volumes, 1984 edition

4 U.S. Army Corps of Engineers, 2006, "Coastal Engineering Manual", Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 Volumes)

5 SNC-Lavalin Inc., 2012, "Lower Churchill Project, Wind-Generated Wave Study", prepared for Nalcor, SLI No. 505573-3001-4HER-0001-00, Nalcor No. MFA-SN-CD-0000-CV-RP-0010-01, 12 September 2012

6 SNC-Lavalin Inc., 2011, “Lower Churchill Project - Design Criteria – Hydraulic”, prepared for Nalcor, SLI No. 505573-300A-4HEC-0001-PC, Nalcor No. M FA-SN-CD-0000-CV-DC-0001-01, 15 November 2011

EXECUTIVE SUMMARY

The following report highlights the methodology and results for the design of riprap protection with respect to wind-generated waves. The methods of the Société d'Énergie de la Baie James and the U.S. Army Corps of Engineers as highlighted in various references have been utilized. The studied structures include the North Spur (upstream and downstream shores), Upstream Cofferdam, Intake Cofferdam, South Dam and Downstream Cofferdam.

For the upstream shore of the North Spur, riprap should range in size from 450 to 800 mm. For all other structures, riprap should range in size from 350 to 600 mm. Detailed results are highlighted in Section 5.7 - Summary of Results. It should be noted that the results and the recommendations are with respect to wind-generated waves. Currently, SLI is carrying out a separate study with regards to riprap design based on ice conditions (SLI No. 505573-3006-40ER-0102, Nalcor No. MFA-SN-CD-0000-CV-RP-0008-01) and the results of that study may take precedence over the impact of wave action on riprap design. Therefore, the results of that study should be reviewed prior to finalizing the design of riprap structures.

1 INTRODUCTION

SNC-Lavalin Inc. has signed an agreement with Nalcor Energy (the Client) to deliver engineering, procurement and construction management services for the Lower Churchill Project (LCP) in Newfoundland and Labrador, Canada.

As part of the LCP, the Muskrat Falls Hydroelectric Development is located on the Churchill River, about 291 km downstream of the Churchill Falls Hydroelectric Development, which was developed in the early 1970's. The installed capacity of the project will be 824 MW (4 units of 206 MW each).

Riprap protection at different locations must be designed such that it can withstand the wave forces at each of the locations listed below. The following report highlights the methodology and results for the design of riprap protection with respect to wind-generated waves only. The methods described will then be applied to:

- North Spur (upstream and downstream shores);
- Upstream Cofferdam;
- Intake Cofferdam;
- South Dam; and
- Downstream Cofferdam.

Figure 1-1 shows the project layout and the locations of the studied structures.

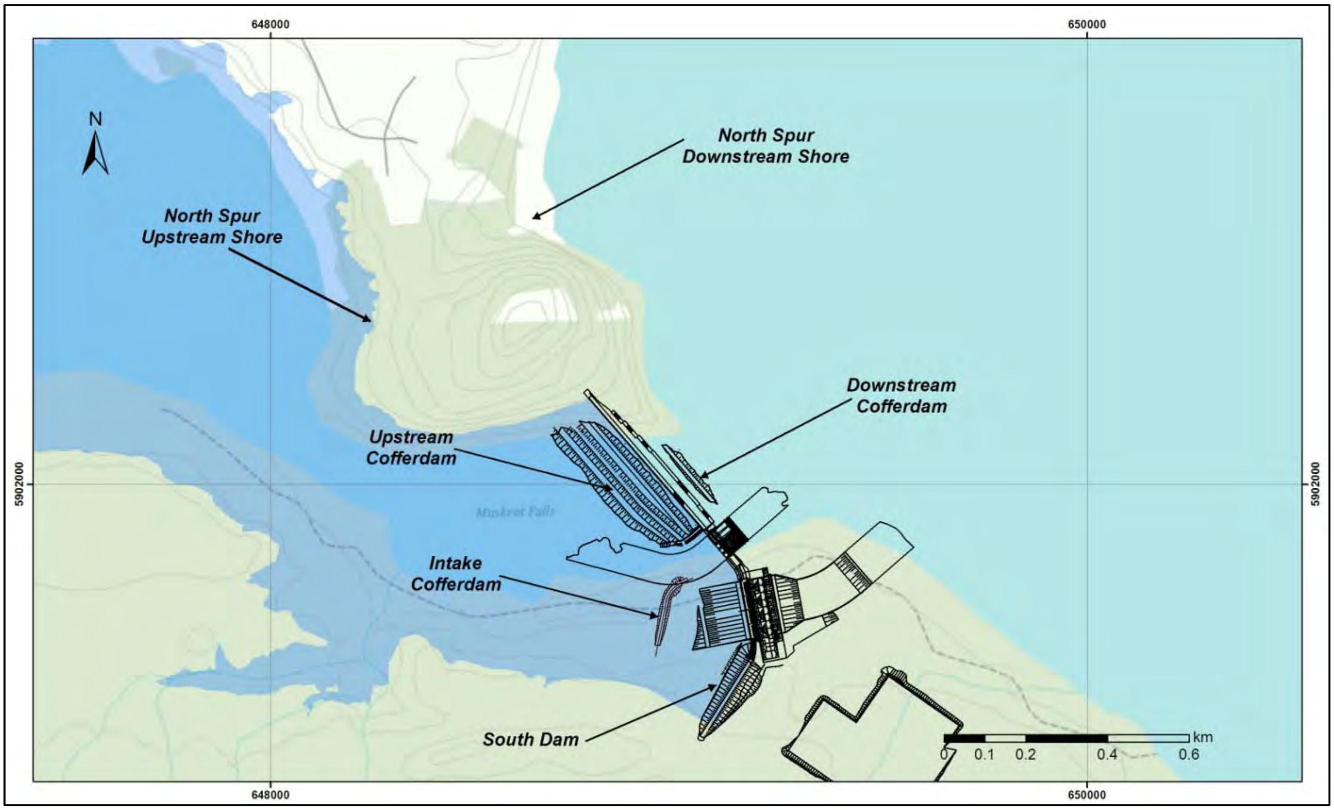

Figure 1-1: Muskrat Falls Hydroelectric Project Layout

2 OBJECTIVES

The objectives of this report are to:

- Present the methodology for the design of riprap protection with regards to wind-generated waves;
- Present the results of the analysis for each structure considered; and
- Make recommendations based on the analysis.

3 SCOPE

The scope of this report is to document the methodology for the design of riprap protection with respect to forces by wind-generated waves for various structures of the Muskrat Falls Hydroelectric Project and present the results of the analysis.

4 METHODOLOGY

The following methodogy is to be followed when designing riprap protection for structures of the Muskrat Falls Hydroelectric Project. These methods follow the procedures of the Société d'Énergie de la Baie James and the U.S. Army Corps of Engineers as highlighted in various references (see Reference List).

4.1 MINIMUM RIPRAP MASS

The Hudson formula is used to evaluate the riprap mass that should resist a certain wave height for specific conditions such as embankment slope and rockfill characteristics. The minimum riprap mass needed to sustain the significant wave is estimated with the following equation (Ref. 2):

$$M_{min} = \frac{\rho_s {H_s}^3}{K\left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha}$$

Where,

M_{min} minimum mass of a single riprap unit (kg)

H_s significant wave height (m)

$\cot\alpha$ cotangent of the angle of the slope with the horizontal

ρ_s mass density of riprap (2,650 kg/m³)

ρ_w mass density of water (1,000 kg/m³)

K stability coefficient

It is recommended to estimate the minimum riprap mass (M_{min}) based on a significant wave for a specific period of recurrence and a stability coefficient (K) equal to 3.5 (Ref. 2). A significant wave with a period of recurrence of 1:1,000 years will be used for all permanent structures (Operation). A significant wave with a period of recurrence of 1:20 years will be used for all temporary structures (Construction).

4.2 REQUIRED GRADATION

The size of the riprap is defined by the minimum and maximum mass. The ratio of the maximum mass to the minimum mass, $\frac{M_{max}}{M_{min}}$, should theoretically not exceed 3.0, however, in the field it is difficult to achieve such a ratio (Ref. 2). In practice, a ratio of 4.0 to 6.0 can be readily achieved and is found to be acceptable, therefore a ratio of 5.0 is used to carry out the computations (Ref. 2).

$$M_{max} = 5.0M_{min}$$

Where,

M_{max} maximum mass of a single riprap unit for design (kg)

4.3 RIPRAP SIZE & QUARRY SELECTION

The dimensions of the riprap are estimated with the following equation (Ref. 2):

$$D_n = \sqrt[3]{\frac{M_n}{C_f \rho_s}}1{,}000$$

Where,

D_n riprap size corresponding to the nth percentage passing (mm)

C_f form coefficient

The minimum and maximum riprap dimensions are calculated based on the minimum and maximum masses. The value of the shape coefficient should be 0.6 (Ref. 2). D_{50} is calculated by taking the average of the maximum and minimum dimensions. The above formula can then be rearranged to determine M_{50}.

Riprap specifications are typically determined by laboratory tests which include density, shape, hardness and durability. The riprap will most likely be selected from gneissic rock formations. Gneiss is a rock with irregular banding which has little tendency to split along planes in contrast with Schist. The rock fragments to be used for riprap shall be selected based on the quality and dimensions of each unit. Fragments should be durable, abrasion resistant and free from seams, cracks and cleavage planes.

Figure 4-1: Gneiss Located in Quarry Q6 on the North Bank of the Churchill River Near Muskrat Falls (Left) Compared to Regular Bands of Schist (Right)

4.4 THICKNESS OF THE RIPRAP LAYER

The thickness of the riprap layer is the greater of 0.30 m (Ref. 3, Ref. 4), or:

$$d_c = 2.0\left(\frac{M_{50}}{\rho_s}\right)^{1/3}$$

or

$$d_c = 1.25\left(\frac{M_{max}}{\rho_s}\right)^{1/3}$$

Where,

d_c thickness of the riprap layer (m)

The design layer thickness is determined by whichever of the above three conditions yields the greatest riprap layer thickness. The specified layer thickness should be increased by 50% for riprap placed underwater.

4.5 EXTENT OF RIPRAP PROTECTION BELOW MINIMUM WATER LEVEL (WL_{MIN})

Typically, riprap protection should extend twice the significant wave height below the WL_{min} (Ref. 2). If this is not possible because the water is too shallow, then appropriate toe berm protection must be put in place.

4.5.1 Toe Berm Protection

The function of a toe berm is to support the main riprap layer and to prevent damage resulting from scour. Riprap displaced from the layer may come to rest on the toe berm, thus increasing toe berm stability. Lamberti (1995) showed that moderate toe berm damage has almost no influence on riprap layer stability. Therefore, in practice it is economical to design toe protection berms that allow for moderate damage.

Practical toe berm stability formulas for waves based on small-scale physical model tests for regular, head-on and oblique waves are as follows (Ref. 4):

$$N_s = \frac{H}{\Delta D_{n50}}$$

Where,

N_s stability number

H wave height in front of the structure, which in this case is equal to H_D (m). H_D is the design wave height and is equivalent to the average of the highest 10% of all waves. This is found by multiplying the significant wave height by 1.27.

Δ $(\rho_s/\rho_w)-1$

ρ_s mass density of riprap (2,650 kg/m³)

ρ_w mass density of water (1,000 kg/m³)

D_{n50} equivalent cube length of toe berm median riprap (m)

The curve in Figure 4-2 below shows the lower boundary of N_s-values associated with acceptable toe berm stability. Some riprap movement would occur; however, the amount of movement is minor and acceptable. This shows that the toe berm isn't overdesigned.

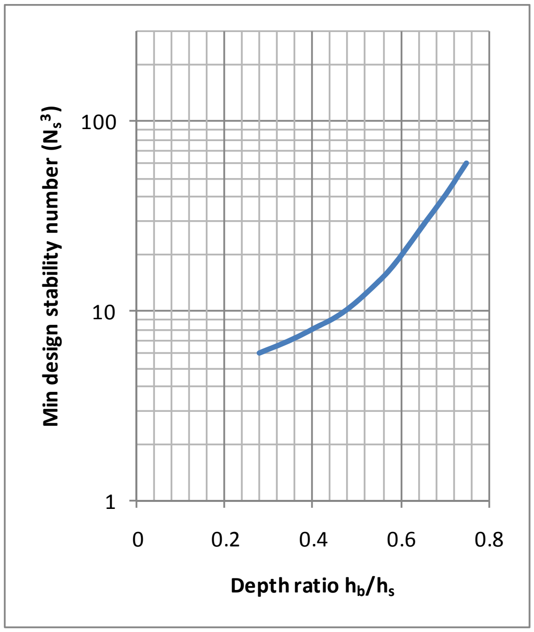

Figure 4-2: Lower boundary of N_s-values associated with acceptable toe berm stability (Ref. 4)

Figure 4-3 is a sketch of a typical cross-section of a riprap toe berm with the parameters defined and is followed by the equation to calculate B.

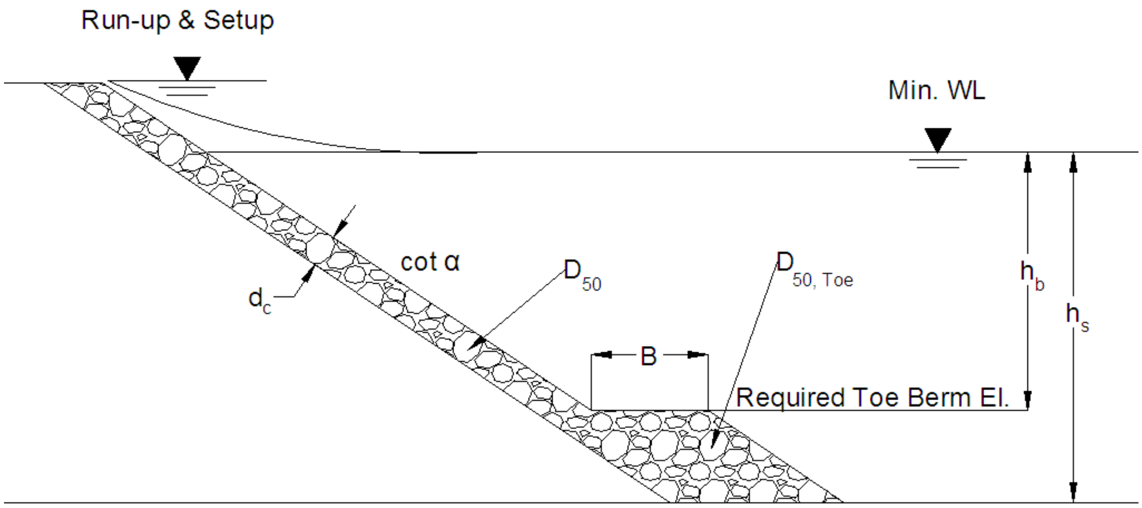

Figure 4-3: Riprap toe berm for slope stability

$$B = 3\left(\frac{W_{50}}{\rho_s}\right)^{1/3}$$

Where,

B length of toe berm (m)

W_{50} average mass of a single riprap unit (kg)

ρ_s mass density of riprap (2,650 kg/m³)

h_b height of the water from the water surface to the top of the toe berm (m)

h_s height of the water from the water surface to the bottom of the water body (m)

Figure 4-4 is a sketch of the cross-section of a riprap toe berm on the downstream shore of the North Spur.

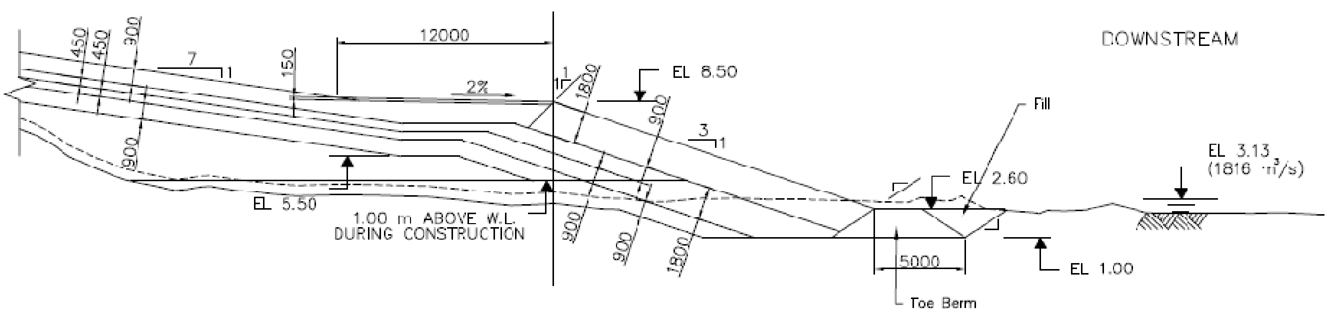

Figure 4-4: Riprap toe berm for slope stability

4.5.2 Breaking Wave Criteria

Design wave conditions for structural stability as well as for functional performance should be chosen based on whether the structure is subjected to the attack of nonbreaking, breaking or broken waves, as well as on the geometrical and porosity characteristics of the structure (Ref. 4). For nonbreaking waves, the design wave height is selected from a statistical distribution and depends on whether the structure is defined as rigid, semirigid, or flexible. For flexible structures, such as riprap structures, the design wave height usually ranges from H_e (the equivalent wave height at the end of the fetch) to H_s (the significant wave height). To determine whether a wave is breaking or nonbreaking, the following formula is applied (Ref. 3):

$$\frac{H}{h_s} > 0.78$$

Where,

H wave height in front of the structure, which in this case is equal to H_D (m)

h_s water depth at the toe of the structure (m)

If $\frac{H}{h_s}$ is greater than 0.78, then the wave is breaking. If it is less than or equal to 0.78, the wave is nonbreaking.

The methodology presented is applicable for non-breaking waves except for the design of the toe berm. Therefore, a verification of the conditions is carried out to confirm the applicability of the method. For the locations with shallow water, a toe berm is considered even for non-breaking conditions.

5 RESULTS

A detailed study on wind-generated waves had been previously carried out (Ref. 5). The results of this study have been extracted for use in the calculations for riprap design.

5.1 NORTH SPUR – UPSTREAM SHORE

The significant wave height from the previous wind-generated wave study (Ref. 5) used in the following calculations for the 1,000-year return period is 1.33 m. The 1:20-year return period is used for design during construction and the significant wave height is 0.98 m. This needs to be considered since during construction a headpond will be created at El. 24 m during the open water season.

The slope of the shoreline on the upstream side of the North Spur varies and calculations were carried out for slopes of 2.5H:1V, 3H:1V, 3.5H:1V, 4H:1V and 5H:1V for the permanent installation. Calculations were carried out for a slope of 2H:1V for the construction phase (1:20-year return period for design).

The results of the analysis are:

Table 5-1: Riprap Design Results – North Spur - Upstream Shore - Construction

Slope	Mass (kg)		
	M_{min}	M_{50}	M_{max}
2H:1V	79	197	397
Slope	**Dimensions (mm)**		
	D_{min}	D_{50}	D_{max}
2H:1V	368	499	629
Slope	**Thickness of Layer (m)**		
	d_c	$d_{c,\ underwater}$	
2H:1V	0.8	1.3	
Slope	**Extent of Protection Below WL_{min} (m)**		
2H:1V	2.0		

Table 5-2: Riprap Design Results – North Spur - Upstream Shore - Operation

Slope	Mass (kg)		
	M_{min}	M_{50}	M_{max}
2.5H:1V	159	395	793
3H:1V	132	329	661
3.5H:1V	113	282	566
4H:1V	99	247	496
5H:1V	79	197	397
Slope	**Dimensions (mm)**		
	D_{min}	D_{50}	D_{max}
2.5H:1V	464	628	793
3H:1V	436	591	746
3.5H:1V	415	562	709
4H:1V	397	537	678
5H:1V	368	499	629
Slope	**Thickness of Layer (m)**		
	d_c	$d_{c,\ underwater}$	
2.5H:1V	1.1	1.6	
3H:1V	1.0	1.5	
3.5H:1V	0.9	1.4	
4H:1V	0.9	1.4	
5H:1V	0.8	1.3	
Slope	**Extent of Protection Below WL_{min} (m)**		
2.5H:1V	2.7		
3H:1V	2.7		
3.5H:1V	2.7		
4H:1V	2.7		
5H:1V	2.7		

The criteria for breaking waves is less than 0.78 for all slopes and waves impacting the upstream shore of the North Spur. Therefore the waves are nonbreaking and the methodology is applicable.

5.2 NORTH SPUR – DOWNSTREAM SHORE

The significant wave height from the previous wind-generated wave study used in the following calculations for the 1,000-year return period is 1.05 m.

The slope of the shoreline on the downstream side of the North Spur was taken as 3H:1V.

The results of the analysis are:

Table 5-3: Riprap Design Results – North Spur - Downstream Shore

Mass (kg)		
M_{min}	**M_{50}**	**M_{max}**
65	162	325
Dimensions (mm)		
D_{min}	**D_{50}**	**D_{max}**
345	467	589
Thickness of Layer (m)		
d_c	**$d_{c,\ underwater}$**	
0.8	1.2	
Extent of Protection Below WL_{min} (m)		
N/A		

The extent of protection below the water level is not applicable in this case since the water depth at the toe is only about 4.0 meters under spring flood with 1:2 year recurrence. Moreover, in winter ice will come into play and a toe berm would protect the stability of the riprap layer. The details for the toe berm protection are as follows:

$$N_s^3 = 10$$

$$D_{n50} = 380 \text{ mm}$$

$$B = 1.2 \text{ m}$$

The criteria for breaking waves is less than 0.78 for waves impacting the downstream shore of the North Spur. Therefore the waves are nonbreaking and the methodology is applicable.

5.3 UPSTREAM COFFERDAM

The significant wave height from the previous wind-generated wave study used in the following calculations for the 20-year return period is 0.70 m.

The slope of the Upstream Cofferdam was taken as 1.6H:1V.

The results of the analysis are:

Table 5-4: Riprap Design Results – Upstream Cofferdam

Mass (kg)		
M_{min}	M_{50}	M_{max}
36	90	181
Dimensions (mm)		
D_{min}	D_{50}	D_{max}
283	384	484
Thickness of Layer (m)		
d_c	$d_{c, underwater}$	
0.6	1.0	
Extent of Protection Below WL_{min} (m)		
1.4		

The criteria for breaking waves is less than 0.78 for waves impacting the Upstream Cofferdam. Therefore the waves are nonbreaking and the methodology is applicable.

5.4 INTAKE COFFERDAM

The significant wave height from the previous wind-generated wave study used in the following calculations for the 20-year return period is 0.70 m.

The slope of the Intake Cofferdam was taken as 1.6H:1V.

The results of the analysis are:

Table 5-5: Riprap Design Results – Intake Cofferdam

Mass (kg)		
M_{min}	M_{50}	M_{max}
36	90	181
Dimensions (mm)		
D_{min}	D_{50}	D_{max}
283	384	484
Thickness of Layer (m)		
d_c	$d_{c,\ underwater}$	
0.6	1.0	
Extent of Protection Below WL_{min} (m)		
1.4		

The criteria for breaking waves is less than 0.78 for waves impacting the Intake Cofferdam. Therefore the waves are nonbreaking and the methodology is applicable.

5.5 SOUTH DAM

The significant wave height from the previous wind-generated wave study used in the following calculations for the 1,000-year return period is 0.95 m.

The slope of the South Dam was taken as 1.7H:1V.

The results of the analysis are:

Table 5-6: Riprap Design Results – South Dam

Mass (kg)		
M_{min}	M_{50}	M_{max}
85	211	425
Dimensions (mm)		
D_{min}	D_{50}	D_{max}
377	510	644
Thickness of Layer (m)		
d_c	$d_{c,\ underwater}$	
0.9	1.3	
Extent of Protection Below WL_{min} (m)		
1.9		

The criteria for breaking waves is less than 0.78 for waves impacting the South Dam. Therefore the waves are nonbreaking and the methodology is applicable.

5.6 DOWNSTREAM COFFERDAM

The significant wave height from the previous wind-generated wave study used in the following calculations for the 20-year return period is 0.91 m.

The slope of the Downstream Cofferdam is 2H:1V.

The results of the analysis are:

Table 5-7: Riprap Design Results – Downstream Cofferdam

Mass (kg)		
M_{min}	M_{50}	M_{max}
64	158	318
Dimensions (mm)		
D_{min}	D_{50}	D_{max}
342	463	585
Thickness of Layer (m)		
d_c	$d_{c,\ underwater}$	
0.8	1.2	
Extent of Protection Below WL_{min} (m)		
N/A		

The extent of protection below the water level is not applicable in this case since the water depth at the toe is only 2.0 meters at construction design spring flood with 1:20 year return period. Moreover, in winter ice will come into play and a toe berm will provide stability to the riprap layer. The details for the toe protection berm are as follows:

$$N_s^3 = 10$$

$$D_{n50} = 330 \text{ mm}$$

$$B = 1.2 \text{ m}$$

The criteria for breaking waves is less than 0.78 for waves impacting the Downstream Cofferdam. Therefore the waves are nonbreaking and the methodology is applicable.

5.7 SUMMARY OF RESULTS

Table 5-8 summarizes the results of the calculations for the studied structures. Figure 5-1 and Figure 5-2 illustrate the terms described in Table 5-8. Figure 5-3 shows the project layout with the calculated D_{50} for each structure.

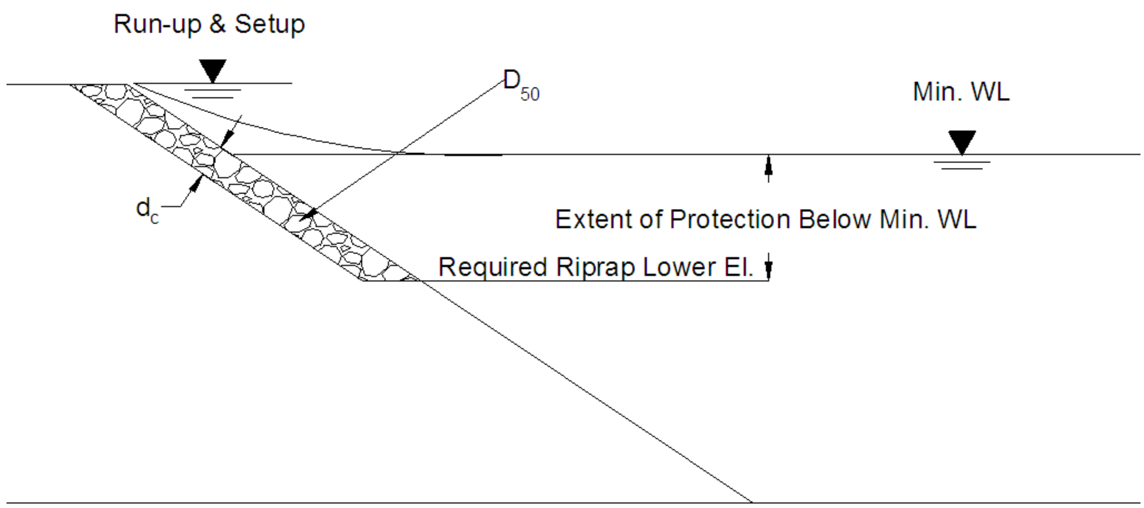

Figure 5-1: Riprap Protection Showing Thickness of Layer and Extent Below Minimum Water Level

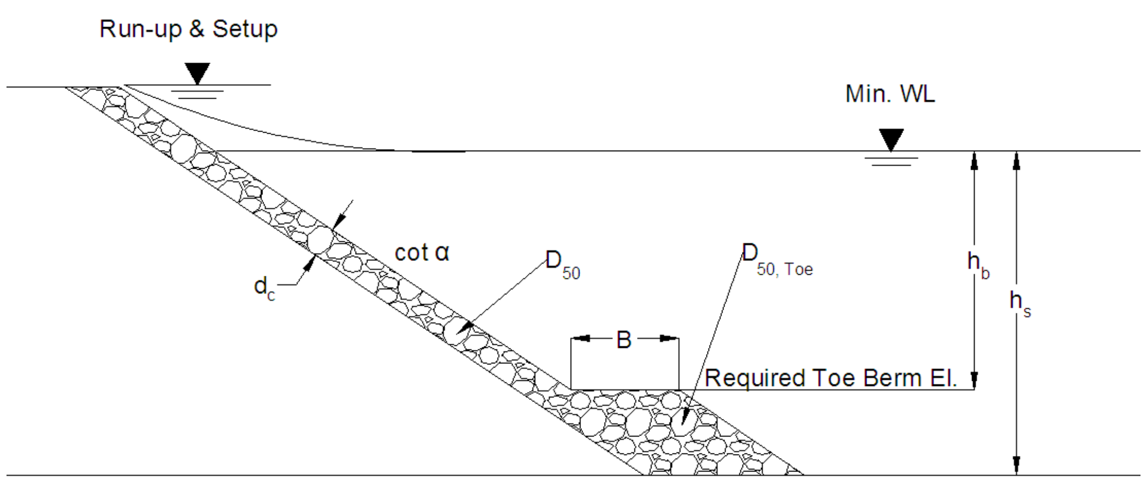

Figure 5-2: Riprap Protection Showing Toe Berm

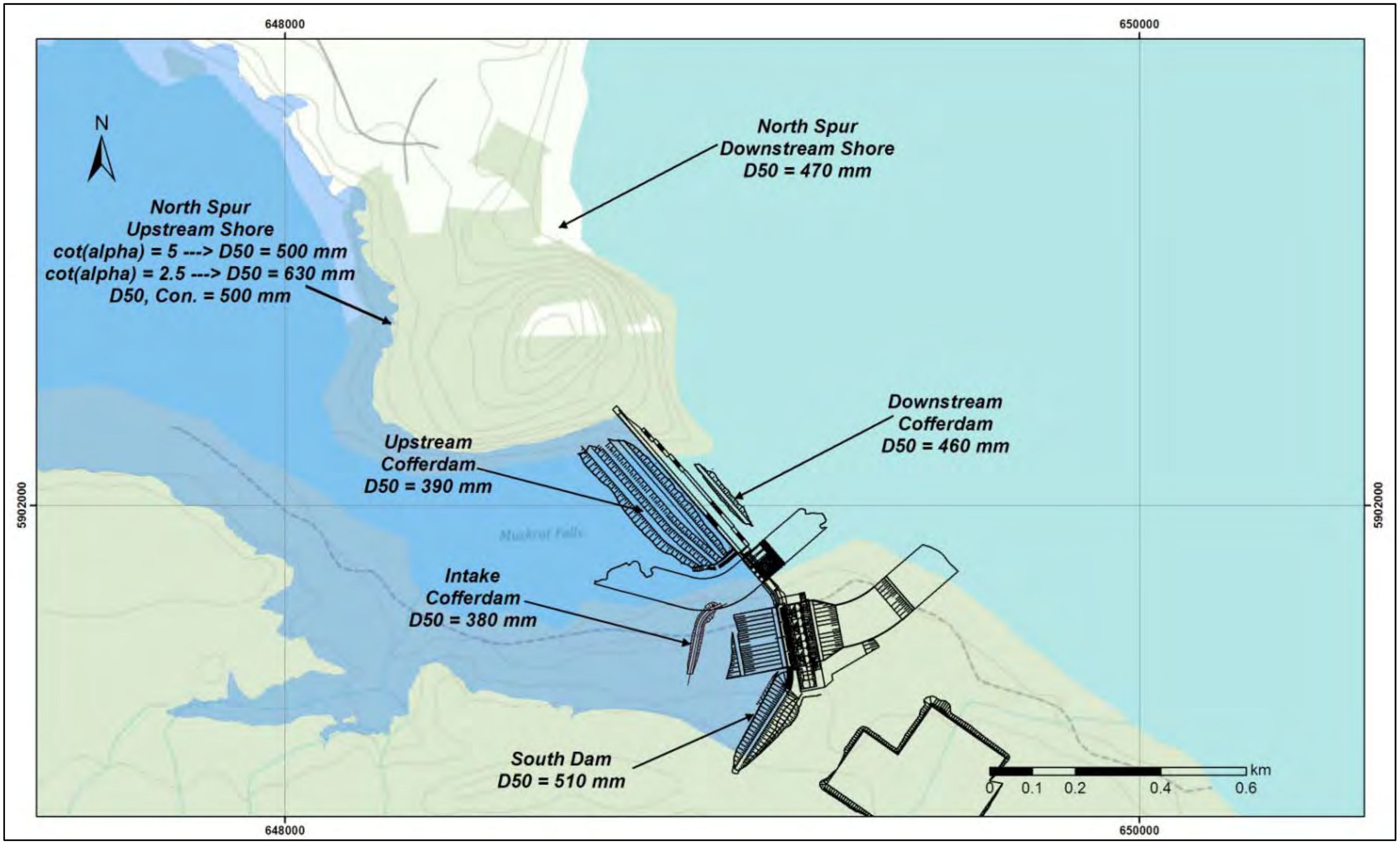

Figure 5-3: Project Layout with Calculated D_{50} for Studied Structures

Table 5-8: Summary of Results

	H_s (m)	cot(α)	D_{min} (mm)	D_{50} (mm)	D_{max} (mm)	Required Thickness d_c (m)	Required Underwater Thickness d_c (m)	Extent of Protection Below WL_{min} (m)	WL_{min} (m)	Required Riprap Upper El. (m)	Required Riprap Lower El. (m)	Depth at Toe, h_s (m)	Required Toe Berm El. (m)	$D_{50, Toe}$ (mm)	B (m)
North Spur Upstream Shore															
Operation	1.33	2.5	460	630	790	1.1	1.6	2.7	38.5	42.9	35.8	34.0	N/A	N/A	N/A
Operation	1.33	3.0	440	590	750	1.0	1.5	2.7	38.5	42.9	35.8	34.0	N/A	N/A	N/A
Operation	1.33	3.5	420	560	710	0.9	1.4	2.7	38.5	42.9	35.8	34.0	N/A	N/A	N/A
Operation	1.33	4.0	400	540	680	0.9	1.4	2.7	38.5	42.9	35.8	34.0	N/A	N/A	N/A
Operation	1.33	5.0	370	500	630	0.8	1.3	2.7	38.5	42.9	35.8	34.0	N/A	N/A	N/A
Construction	0.98	2.0	370	500	630	0.8	1.3	2.0	24.0	26.5	22.0	20.0	N/A	N/A	N/A
North Spur Downstream Shore															
Operation	1.05	3.0	350	470	590	0.8	1.2	N/A	1.3	7.5	N/A	4.0	3.0	380	1.2
Upstream Cofferdam															
Construction	0.70	1.6	280	380	480	0.6	1.0	1.4	24.0	26.0	22.6	20.0	N/A	N/A	N/A
Intake Cofferdam															
Construction	0.70	1.6	280	380	480	0.6	1.0	1.4	24.0	26.0	22.6	20.0	N/A	N/A	N/A
South Dam															
Operation	0.95	1.7	380	510	640	0.9	1.3	1.9	38.5	46.3	36.6	34.0	N/A	N/A	N/A
Downstream Cofferdam															
Construction	0.91	2.0	340	460	590	0.8	1.2	N/A	1.3	7.9	N/A	2.0	5.0	330	1.2

6 RECOMMENDATIONS

Based on the results, two gradations of riprap can be considered for design and are shown in Table 6-1. The required D_{50} and riprap protection extents for each location is shown on Figure 6-1. It should be noted that the results and the recommendations are with respect to wind-generated waves only. Currently, SLI is carrying out a separate study with regards to riprap design based on ice conditions (SLI No. 505573-3006-40ER-0102, Nalcor No. MFA-SN-CD-0000-CV-RP-0008-01) and the results of that study may take precedence over the impact of wave action on riprap. The results of that study should be reviewed prior to finalizing structure design.

Table 6-1: Riprap Gradations to Be Considered for Design

Riprap Gradation Size (mm)	Structure	Recommendation
280 – 480	Upstream Cofferdam - Construction Intake Cofferdam - Construction	D_{min} = 400 mm D_{max} = 600 mm d_c = 0.9 m B = 1.3 m* $D_{50,\ Toe}$ = 500 mm*
350 – 600	North Spur (D/S Shore) – Construction & Operation Downstream Cofferdam - Construction	
380 – 640	North Spur (U/S Shore) – Construction South Dam - Operation	
460 - 800	North Spur (U/S Shore) – Operation	D_{min} = 400 mm D_{max} = 800 mm d_c = 1.2 m

*Only if a toe berm is required. This is applicable for only the Downstream Cofferdam and downstream shore of the North Spur.

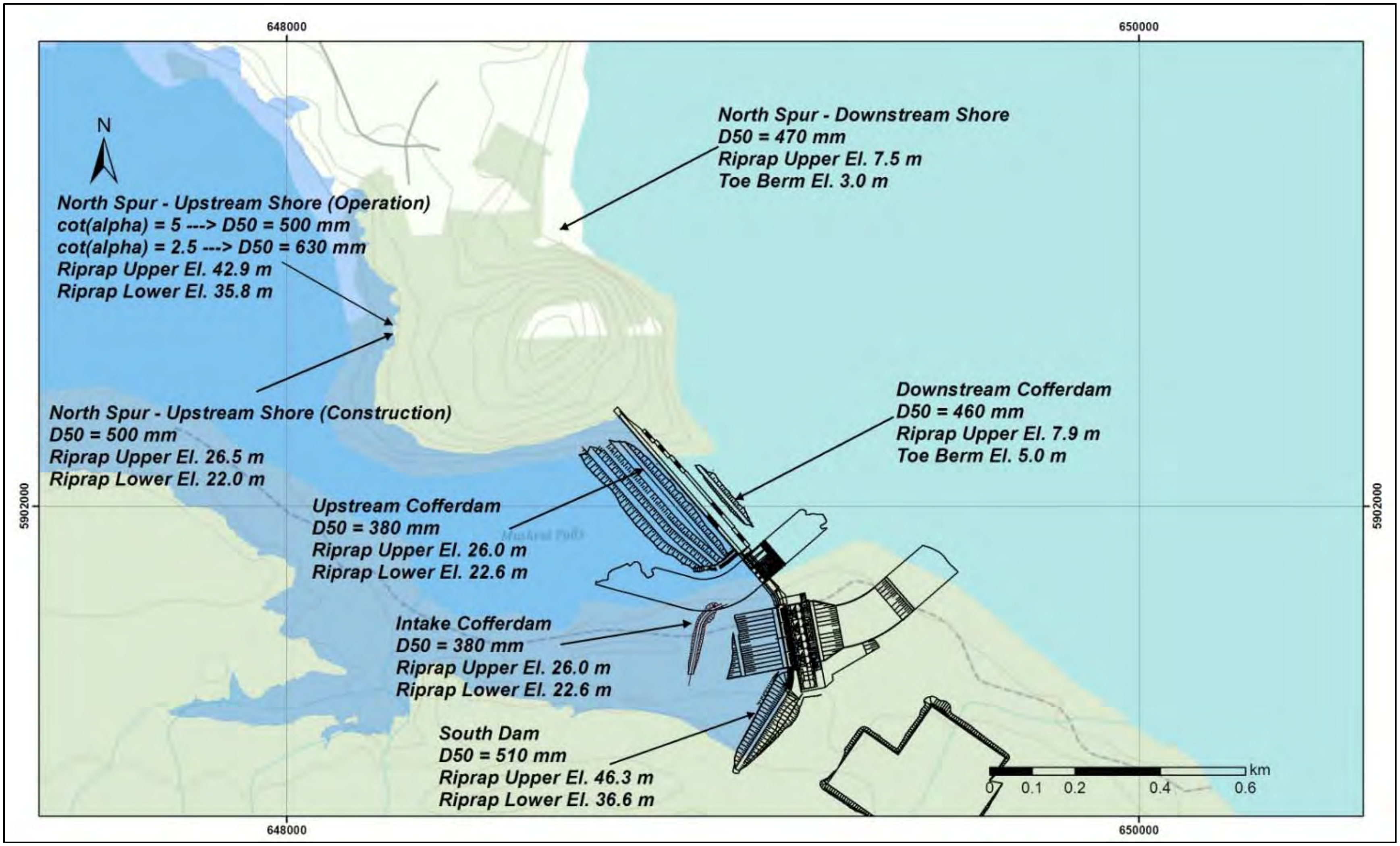

Figure 6-1: Final Recommended D_{50} and Protection Elevation Extents for Studied Structures