

To:	Paul Bosetti, Chief Administrative Officer	Date:	August 12, 2024
cc:	Graham Wilkins, Otavio Sayao	Memo No.:	002
From:	Francis Edomwonyi, P.Eng.	File:	704-TRN.VHWY03300-01
Subject:	Hamlet of Chesterfield Inlet – Oceans Protection Plan Sealift Improvements Project – Design Update Memo, August 12, 2024		

1.0 INTRODUCTION / PURPOSE

Tetra Tech Canada Inc. (Tetra Tech) is pleased to provide this status update memo to the Hamlet of Chesterfield Inlet regarding the Oceans Protection Plan (OPP) project to upgrade the Community's existing sealift facility.

The purpose of this memo is to:

- Summarize work completed to date.
- Provide estimated material quantities for the access road, ramp and laydown area. Note different options for the ramps are presented.
- Outline next steps on the project and Community input required to progress the design.
- Define the risk level acceptable to the community, which is called 'encounter probability' of the loading condition, which may be estimated based on the return period of the design waves and the expected lifetime of the shoreline works.

We have conducted a review of standard practice for the province of Alberta and a municipality with similar characteristics for the design life of 50 years. The practice is to design for 50-year design life with a return period of 50 year. This would result in likelihood of 50-year event to occur at 45 percent. To further reduce the likelihood, the cost would be significantly higher.

Tetra Tech is available to meet virtually to discuss the contents of this technical memo and the next steps on the project.

2.0 WORK COMPLETED TO DATE

Tetra Tech has completed the following on the project to date:

- Collected and reviewed background information.
- Conducted Engineer's Site Visit on August 21-24, 2023, which included discussions / interviews with Community representatives.
- Tested soil / substrate and water samples collected during the site visit.

- Contacted barge operators and received input regarding their operational requirements, issues with the existing sealift facility, and desired improvements.
- Completed hydrotechnical Analysis for the preferred Laydown area option I.
- Conceptual design for the preferred Laydown area option I, sealift ramp and access roads

Further details of the completed hydrotechnical analysis are provided in Appendix B.

2.1 Construction Typical Sections and material types

2.1.1 Laydown Area

It is envisaged that the laydown area will be constructed with the following materials:

- Embankment material – Earth
- Subbase material
- Crushed gravel – 19 mm
- Boulders : the existing boulders in the intertidal area and around the community will be utilized. These boulders are not adequate to be used as armour or Riprap (slope protection) to the laydown area as they have moved due to ice action and have scattered in the nearshore area.
- Rip Rap : The armour (slope protection) layer of Riprap to the laydown area is designed to withstand ice action, so it should not move, except if ice thickness is larger than what our design was based on. The armour stones are not likely to migrate to the sealift ramp as the riprap will wrap around the laydown area completely. The riprap can be obtained from quarries, for instance a $D_n = 1$ m corresponds to a stone mass of about 2.6 tonnes, which is possible to obtain from quarries.
- Woven geotextile

Details of the conceptual typical cross section of the laydown area is provided in Figure 1.

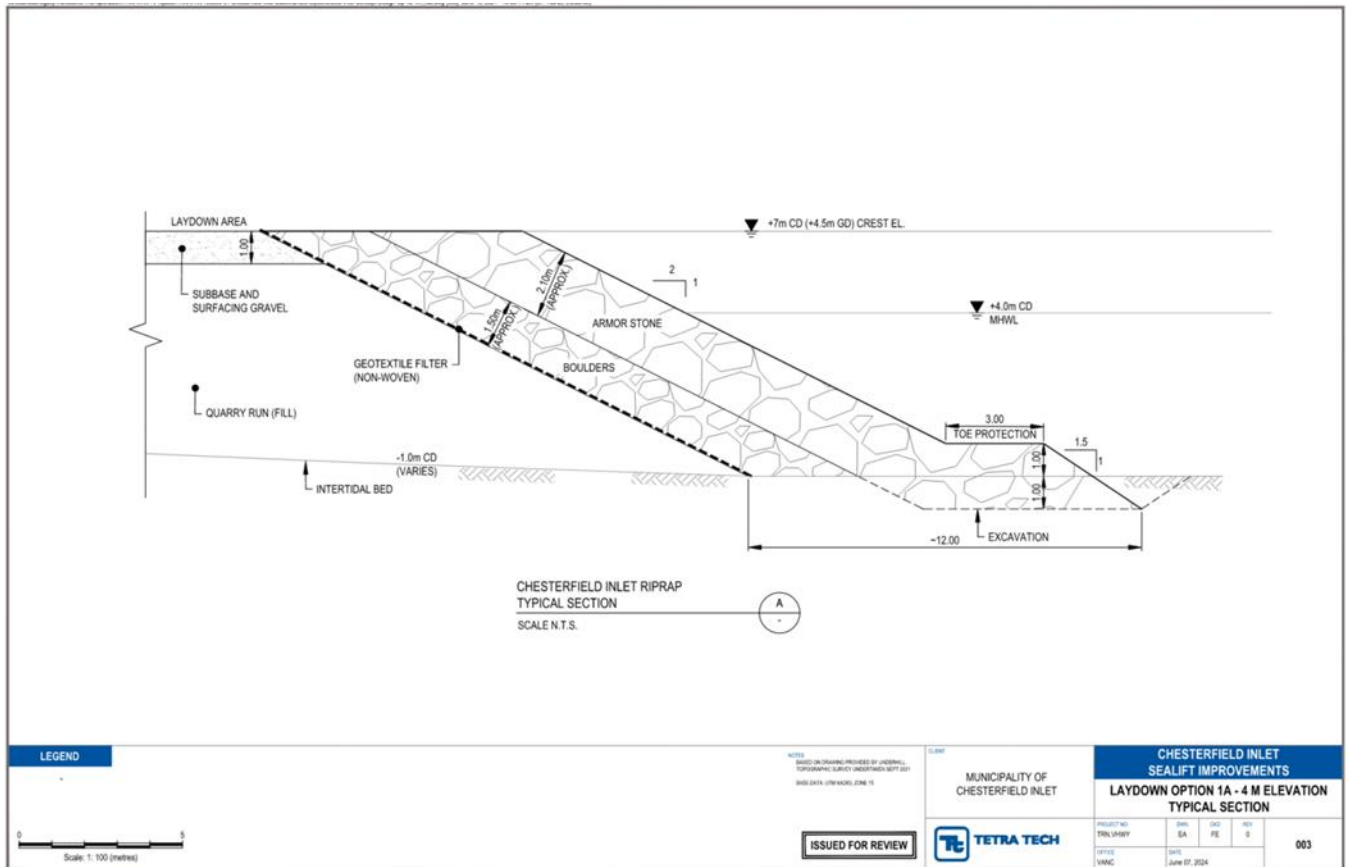


Figure 1: Laydown Area Typical Conceptual Cross Section

2.1.2 Sealift Ramp

It is envisaged that the sealift ramp will compose of the following construction materials:

- 19 mm of crushed base gravel.

Details of the conceptual typical cross section of the laydown area is provided in Figure 2.

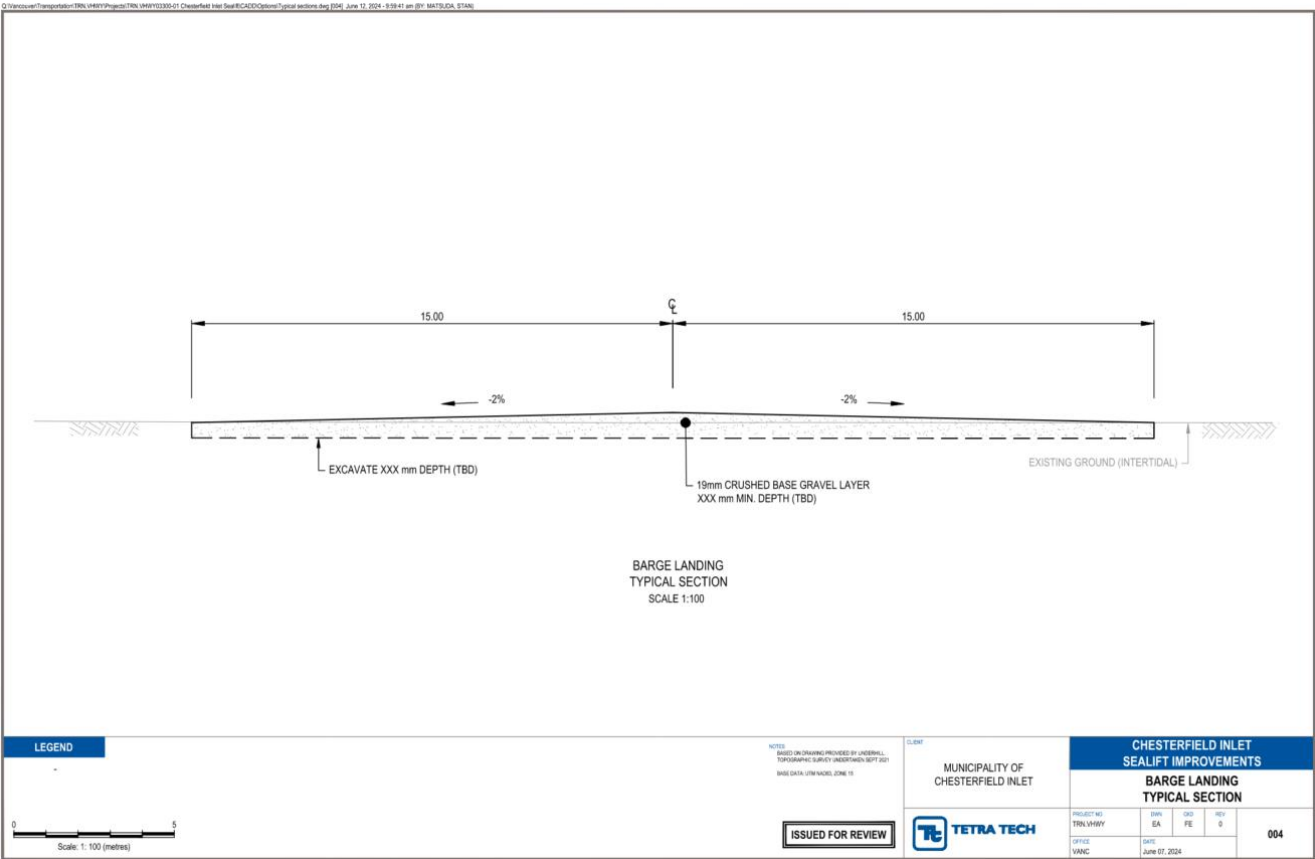


Figure 2: Conceptual Barge Landing Typical Cross section.

2.1.3 Access Road

It is envisaged that the access road will compose of the following construction materials:

- 19 mm minus crushed base gravel.

Details of the conceptual typical cross section of the laydown area is provided in Figure 3.

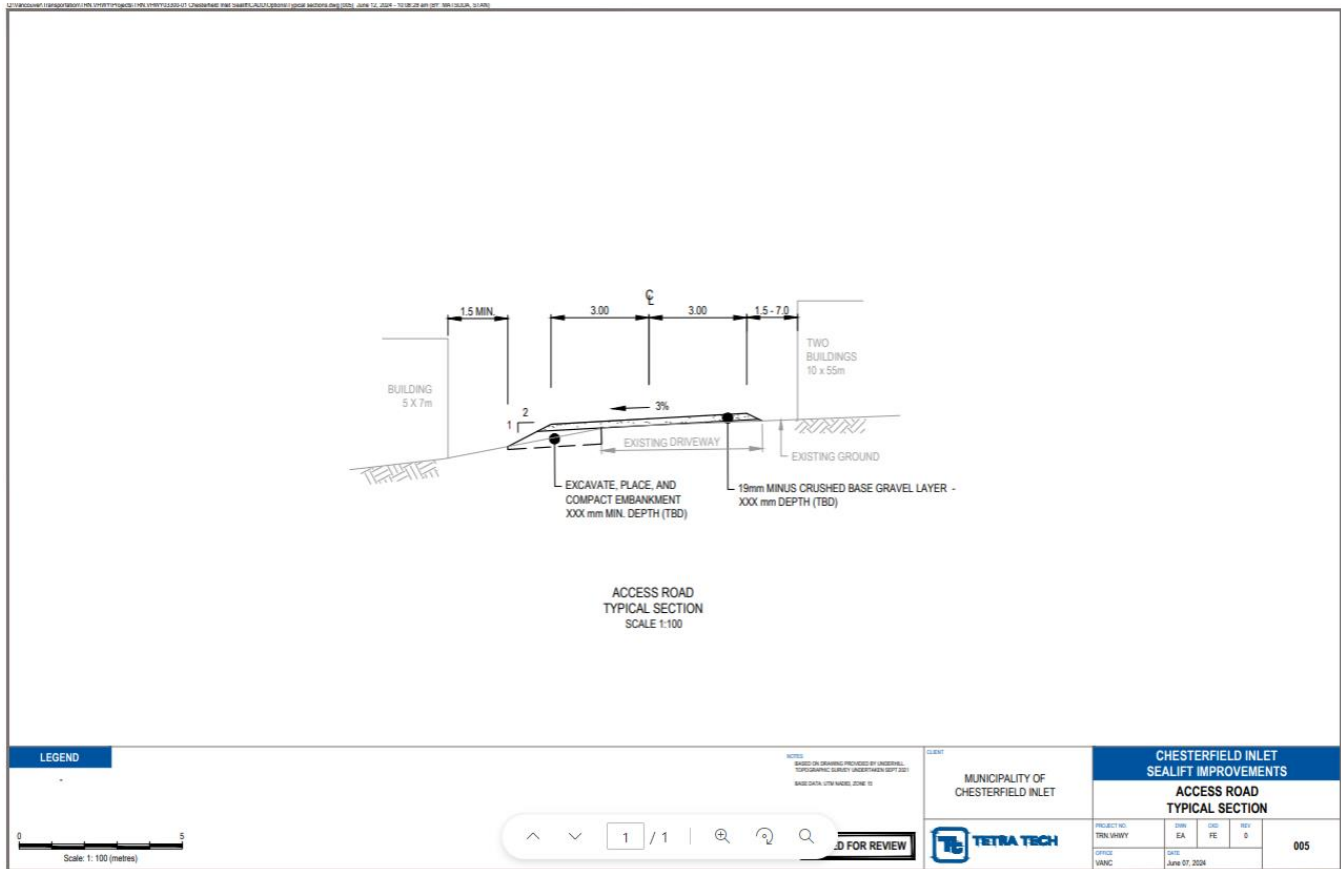


Figure 3: Conceptual Access Road Typical Cross section.

3.0 COMMUNITY INPUT

3.1 Community Input – Laydown Area

To progress the design to the construction stage, Tetra Tech requires the following input from the Community:

3.1.1 Confirm Level of Risk – Elevation for Laydown Area Fill

- Define the risk level acceptable to the community, which is called 'encounter probability' of the loading condition, which may be estimated based on the return period of the design waves and the expected lifetime of the shoreline works.
- We have conducted a review of standard practice for the province of Alberta and a municipality with similar characteristics for the design life of 50 years. The practice is to design for 50-year design life with a return period of 50 year. This would result in likelihood of 50-year event to occur at 45 percent. To further reduce the likelihood, the cost would be significantly higher.

3.1.1.1 Options with Climate Change

Our hydrotechnical analysis presents three scenarios for the Laydown area pad (fill) elevation **considering the influence of climate change and sea level rise (SLR)**, depending on the return period selected for the design waves at the site.

The buildings that are in higher grounds, between the laydown area and the existing sealift ramp, are located in elevations between +3.5 m GD to +4.0 m GD.

The elevations obtained from the hydrotechnical analysis for the laydown area fill for a lifetime of 30 years depending on the return period and the probability of occurrence during the structure lifetime are as shown below.

Option	Return Period (Years)	Laydown Elevation (m GD)	Probability of Occurrence (%)
1	2	+3.5	100
2	10	+4.0	96
3	50	+4.5	45

Foot note

Options without Climate Change

The three options for the laydown area pad (fill) elevation **without considering the impact of climate change and sea rise level**, for a lifetime of 30 years for the laydown area, will decrease by 0.2m for option 1, 0.5m for option 2 and 0.3m for option 3.

This information has been provided to show the impact of climate change and sea rise on the design.

3.1.1.2 Discussions

The community may choose the design elevation option they prefer, which is directly linked to the acceptable risk. The main buildings are located in elevations between +3.5 m GD and +4 m GD, corresponding to two of our options. Some smaller buildings near the sealift ramp are in lower elevations.

Our option with RP = 50 years is 0.5 m higher than the main buildings, at +4.5 m GD, which is the elevation of the road.

If design elevations are lower, say at +3.5 m GD, then this elevation is covered with our option with RP = 2 years. In such case, there is potential that waves will overtop (partial flood) the laydown area, and containers may experience some wet surfaces, on certain situations, anticipated to be occurring every other year.

The encounter probability for a design wave of $H_s = 1.2$ m and a RP = 2 years (elevation of +3.5 m GD), ~~that~~ means that during the assumed 30 years of lifetime of the laydown area pad there is 100% of chance of the occurrence of the design wave, i.e., the laydown area experiencing some wave runoff/flood. This event may occur immediately after construction of anytime for 30 years. It may also occur (likely) more than once.

The community indicated no major concern with flooding, but past waves did overtop some of their existing areas, which is in agreement with our wave modelling results.

3.1.1.3 Conclusion

Option 1 laydown crest elevation +3.5 m GD (Geodetic Datum)

- Storm events with a return period of up to once in 2 years will reach the crest (top) elevation but not overtop the laydown area. There is a 100% chance (probability) of the design wave climate to occur during the 30-year design life of the laydown area pad.
- All areas in the community that are constructed at +3.5 meters or less will experience design wave runup and potentially some minor flooding within the 30-year design life. See Figure 1.
- Volume of quarry material required will be 48,300 m³. The high-level cost will be \$5,364,000 (Does not include contingency)

Option 2 laydown crest elevation +4.0 m GD (Geodetic Datum)

- Storm events with return period of up to once in 10 years will reach the crest (top) elevation but not overtop the laydown area. There is a 96% chance of the design wave climate to occur during the 30-year design life of the laydown area pad.
- All areas in the community that are constructed at +4.0 meters or less will experience design wave runup and potentially some minor flooding within the 30-year assumed design life. See Figure 1.
- Volume of quarry material required will be 55,330 m³. The high-level cost will be \$5,939,450 (Does not include contingency)

Option 3 laydown crest elevation +4.5 m GD (Geodetic Datum)

- Storm events with return period of up to once in 50 years will reach the crest (top) elevation but not overtop the laydown area. There is a 45% chance of the design wave to occur during 30-year assumed design life of the laydown area pad.
- All areas in the community that are constructed at +4.5 meters GD or less will experience design wave runup and potentially some minor flooding within the 30-year assumed design life. See Figure 1.
- Volume of quarry material required will be 63,530 m³. The high-level cost will be \$6,605,405 (Does not include contingency)

Footnote:

- 2021 Inuvialuit Energy security Project unit rate multiply by 1.15 (assuming 5% annual inflation)

Considerations for the Community

- The Community may define the risk level they consider acceptable, which is estimated based on the return period of the design waves and the expected lifetime of the works. As design life of the proposed laydown area pad and shoreline protection works, Tetra Tech considered a 30-year life.
- To reduce the chance (probability) of a storm event with a return period of once in 50 years, to occur during the assumed design lifetime of the laydown area, the pad would need to be built to a +4.5 m GD crest elevation.

- The higher the design crest elevation of the laydown area, the smaller is the chance of the design wave conditions to occur during the laydown area pad lifetime. We also note that while this higher elevation would better protect the laydown area, the surrounding community buildings will not be flooded unless they are located at a ground elevation lower than approximately +3.5 m GD.

The Community need to define the risk level they consider acceptable. That is the elevation that corresponds to the risk level, before the final design can commence.



Figure 1 : Conceptual Project Plan

4.0 COMMUNITY INPUT AND NEXT STEPS

Once the Community confirm the level of risk for the Laydown Pad elevation, Tetra Tech will undertake the following tasks to progress the design:

- Geotechnical Investigation and Analysis
- Finalizing Laydown pad elevation based on the geological findings.
- Prepare design drawings and construction cost estimate for review by the Community, including design drawings for the sealift ramp improvements (GA, plan / profile, typical sections and details, cross sections).
- Prepare and submit regulatory applications (NIRB, DFO).
- Develop construction tender package, including detailed design drawings, contract specifications, construction cost estimate, unit price table and contract form (assumed to be CCDC).

5.0 UPDATED PROJECT SCHEDULE

Tetra Tech's updated schedule for the remaining project scope is summarized in Table 7-1. Tetra Tech will continue to update the project schedule as the design progresses.

Table 5-1: Tetra Tech's Updated Schedule

Task	Milestone Completion Date
Confirmation of Community's Preferred Risk Level	August 26, 2024
Geotechnical Investigation and analysis	October 30, 2024
Detailed Design and Tender Package Preparation	December, 2024
Regulatory Approvals	April 2025 (estimated)
Tender Support Services (pending receipt of regulatory approvals)	Spring 2025
Construction (pending receipt of regulatory approvals)	Summer 2025

6.0 NEXT STEPS

Once the Community confirms the level of risk for the Laydown Pad elevation, Tetra Tech will undertake the following tasks to progress the design:

- Geotechnical Investigation and Analysis
- Finalizing Laydown pad elevation based on the geological findings.
- Prepare design drawings and construction cost estimate for review by the Community, including design drawings for the sealift ramp improvements and access road (GA, plan / profile, typical sections and details, cross sections).
- Prepare and submit regulatory applications (NIRB, DFO).

7.0 LIMITATIONS OF REPORT

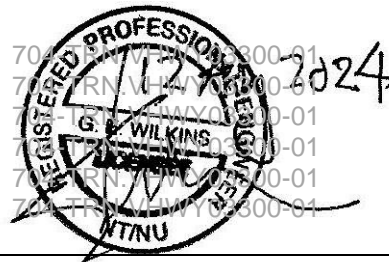
This report and its contents are intended for the sole use of the Hamlet of Chesterfield Inlet and their agents. Tetra Tech Canada Inc. (Tetra Tech) does not accept any responsibility for the accuracy of any of the data, the analysis, or the recommendations contained or referenced in the report when the report is used or relied upon by any Party other than the Hamlet of Chesterfield Inlet, or for any Project other than the proposed development at the subject site. Any such unauthorized use of this report is at the sole risk of the user. Use of this document is subject to the Limitations on the Use of this Document attached in Appendix D or Contractual Terms and Conditions executed by both parties.

8.0 CLOSURE

We trust this technical memo meets your present requirements. If you have any questions or comments, please contact the undersigned.

Respectfully submitted,
Tetra Tech Canada Inc.

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Date	12 AUGUST 2024
PERMIT NUMBER: P 018	
NT/NU Association of Professional Engineers and Geoscientists	

/FE
Enclosure: Limitations on the Use of this Document

APPENDIX A

MOTI 2020, SECTION 205 RIPRAP

SECTION 205
RIPRAP

205.01 General – This Section covers the protection of embankments and channels, by using the Class of riprap, at the designated locations, specified in the Drawings and Special Provisions.

Work within any watercourse shall be carried out in conformity with the environmental protection provisions in the Contract and SS 165, all to the satisfaction of the Ministry Representative.

205.02 Quality Control

205.02.01 General – The Contractor shall be fully responsible for all quality control testing, inspection and documentation to achieve compliance with the Contract.

205.02.02 Material Testing

- (a) **Initial Testing and Gradation** – The Contractor shall provide documented test results for the riprap material properties, per Table 205-A, and the gradation of the riprap to the Ministry Representative for review at least one week prior to starting riprap delivery to the Site.
- (b) **Gradation Sampling Areas** – For riprap gradation, quality control shall continue to occur throughout the project and include, but not be limited to the following:
- (i) gradation of representative samples of riprap at the source while production is underway; and
 - (ii) gradation of the placed riprap at the Site while placement is underway and at completion of the placement.
- (c) **Gradation Sampling Frequency** – Gradation quality control shall occur at a frequency of not less than once per week, or once per 1000 tonnes or 500 m³ of riprap produced or placed, whichever is more frequent, at each source and placement site that is active at any time during the week.

205.02.03 Gradation Control – The Contractor may elect to perform gradation quality control using either mass or dimensional assessment in accordance with the following.

- (a) **Light Riprap** – For riprap of Class 50 or less, quality control may be done visually, supplemented with measuring dimensions or masses as necessary to confirm the accuracy of the visual assessment.
- (b) **Heavy Riprap by Mass** – The quality control for the gradation of the Class 100 and heavier riprap by mass shall:
- (i) Conform to the gradation specified in using Table 205-B ; and
 - (ii) be done in accordance with the ASTM D5519 Method A or Method C.

Table 205-A: Tests for Riprap Material Properties

Property	ASTM Test Designation	Allowable Value
Specific Gravity	D6473	≥2.50
Absorption	D6473	≤2%
Soundness by use of Magnesium Sulphate	D5240	≤10% (following 5 cycles)
Micro-Deval Abrasion Loss Factor	D6928	≤20%

- (c) **Heavy Riprap by Dimension** – The quality control for the gradation of Class 100 and larger riprap by size shall:
- (i) Conform to the gradation specified in Table 205-C with a tolerance of -5% to +15 % of the values listed; and
 - (ii) be done using ASTM D5519 Method B, the [FHWA FLH T 521](#) Wolman count method, or other method approved in advance by the Ministry Representative.
- (d) **Documentation** – Documentation of the gradation testing and inspection shall be provided to the Ministry Representative within 24 hours from its completion.

205.02.04 Hold Points – The following hold points, as defined in SS 145.12, will apply for Class 100 kg riprap and larger unless otherwise directed by the Ministry Representative:

- Gradation QC and material property test results for riprap are provided to the Ministry Representative prior to starting delivery to Site
- Completion of Toe/Terminal end key excavations
- Preparation of back slope/surface
- Application of filter(s)
- Completion of toe construction

205.03 Quality Assurance – The Ministry may carry out quality assurance by auditing the Contractor's quality control program and by testing and inspection at its discretion. Ministry quality testing shall not relieve the Contractor of responsibility for providing quality control.

205.04 Riprap Properties

205.04.01 General - Rock shall be hard, durable, and angular quarry rock of a quality that will not disintegrate on exposure to water or the atmosphere.

205.04.02 Size and Gradation – The size and gradation of rocks shall be in accordance with the following:

- well-graded;
- Table 205-B for mass or Table 205-C for dimensions; and
- the minimum dimension of each individual rock shall be greater than one-third of its maximum dimension;

205.04.03 Material Properties – Rocks used for riprap shall only break with difficulty, have no earthy odour, no closely spaced discontinuities, and should not absorb water easily. Rocks composed of appreciable amounts of clay or silt shall not be accepted for use as riprap.

Any riprap source shall be tested by the Contractor for conformance to the requirements of Table 205-A, prior to use on the Site.

Where the Ministry has provided a riprap source and has previously tested its material properties, the Contract will indicate that, and the Contractor will not be required to undertake further material property testing.

Where the Contractor elects to use another source, or the Ministry does not have test results for an available Ministry source, the Contractor will be responsible to test to, and ensure compliance with, the allowable values for the tests given in Table 205-A.

Representative samples may be broken off representative riprap rocks and crushed to allow performance of the tests.

Rocks shall be tested for Acid Rock Drainage and Metal Leaching as required by the Ministry Representative. Testing shall meet the Ministry requirements outlined in the Ministry's [Technical Circular T-04/13](#), available at the link below.

<https://www2.gov.bc.ca/assets/gov/driving-and-transportation/transportation-infrastructure/engineering-standards-and-guidelines/technical-circulars/2013/t04-13.pdf>

205.05 Preparation – Areas to receive riprap shall be trimmed to a uniform surface and to the slope(s) indicated on the Drawings or as directed by the Ministry Representative.

Before rock placement commences, loose material shall be removed and minor hollows filled with surrounding native materials well tamped-in to the approval of the Ministry Representative.

Table 205-B: Gradation of Rock by Class of Riprap

Class of Riprap (kg)	Rock Mass (kg)			
	Percentage Smaller Than Given Rock Mass			Max. Size
	15%	50%	85%	
10	1	10	30	50
25	2.5	25	75	125
50	5	50	150	250
100	10	100	300	500
250	25	250	750	1 250
500	50	500	1 500	2 500
1000	100	1 000	3 000	5 000
2000	200	2 000	6 000	10 000
4000	400	4 000	12 000	20 000

Table 205-C: Gradation and Intermediate Dimension of Rock by Class of Riprap

Class of Riprap (kg)	Intermediate Dimension (mm)			
	Percentage Smaller Than Intermediate Dimension			Max. Size
	15%	50%	85%	
10	90	200	285	350
25	125	270	385	450
50	155	340	485	600
100	200	425	610	750
250	270	575	830	1 000
500	340	725	1 050	1 250
1000	425	915	1 325	1 600
2000	535	1 150	1 650	2 000
4000	675	1 450	2 100	2 500

Note: Table 205-C shows the intermediate dimension as defined in the Wolman method as per [FHWA FLH T 521](#) corresponding to the rock mass shown in Table 205-B, based on spherical volume, using Specific Gravity = 2.50. Regardless of actual source Specific Gravity, the dimensions indicated remain applicable (subject to the limits specified in Table 205-A).

205.06 Filter Layers – Filter layers and placement, where required, shall be as per the Drawings and Special Provisions or as directed by the Ministry Representative.

205.07 Foundations and Placement – The nominal thickness and the surface width for each Class of riprap shall conform with the requirements specified in Table 205-D.

Each truckload of rock brought to the site shall provide a complete range of the rock sizes in the gradation.

Work shall be carried out to prevent cracking or breaking of rock riprap by crushing under machine tracks. Work shall be carried out to avoid disturbing the filter layer(s). Damage shall be repaired at the Contractor's expense.

Placement shall be as indicated on the Drawings or as directed by the Ministry Representative.

Layout and installation details for riprap at bridge embankments and for embankment protection works parallel to waterway flow shall be in accordance with SS Drawing SP205-1, unless otherwise specified. SS Drawing SP205-1 shall not apply for riprap for culvert end treatment or around in-stream piers.

At the toe of sloped riprap, a sufficient number of the larger rocks shall be placed to form a firm foundation. The remaining larger rocks shall be distributed evenly throughout the mass.

Rocks shall be placed to the required thickness, providing a reasonably well-graded mass with the minimum of voids. Clusters of small or large stones shall be avoided.

205.08 Machine Placed Riprap – The controlled placement of rock of the Class specified shall produce a well-graded rock mass of the nominal or required thickness over the area indicated. Placement of riprap shall not be by end-dumping. The rock shall be machine manipulated as necessary to provide mass stability and a regular surface with a minimum of voids.

205.09 Hand-Laid Riprap – Hand-laid riprap, normally Class 10 or 25, shall conform to the size, gradation and requirements set out in SS 205.04. Individual rocks too large to handle shall be machine manipulated for satisfactory setting and spacing.

205.10 Grouted Riprap – Where grouted riprap is shown or required, the surfaces of the rocks shall be cleaned and wetted and the interstices filled with cement mortar, well rodded and pounded in for a minimum mortar depth of 300 mm or as otherwise detailed or required by the Ministry Representative. The mortar shall consist of one-part Portland cement to three parts well-graded clean fine aggregate (1:3) mixed to a proper consistency.

Table 205-D: Placement Dimensions by Class of Riprap

Class of Riprap (kg)	Nominal Thickness of Riprap* (mm)	Surface Width, W* (mm)	
		2H:1V Slope	1.5H:1V Slope
10	350	783	631
25	450	1006	811
50	550	1230	992
100	700	1566	1262
250	1000	2236	1803
500	1200	2684	2163
1000	1500	3355	2704
2000	2000	4473	3606
4000	2500	5591	4507

* See SS Drawing SP205-1 for the description of the Nominal Thickness and Surface Width dimension "W".

MEASUREMENT

205.11 Measurement – Measurements will be made by multiplying the facial area by the average thickness dimensions as shown on the Drawings or as directed by the Ministry Representative. No allowance will be made for the quantity of rock placed in excess of these dimensions.

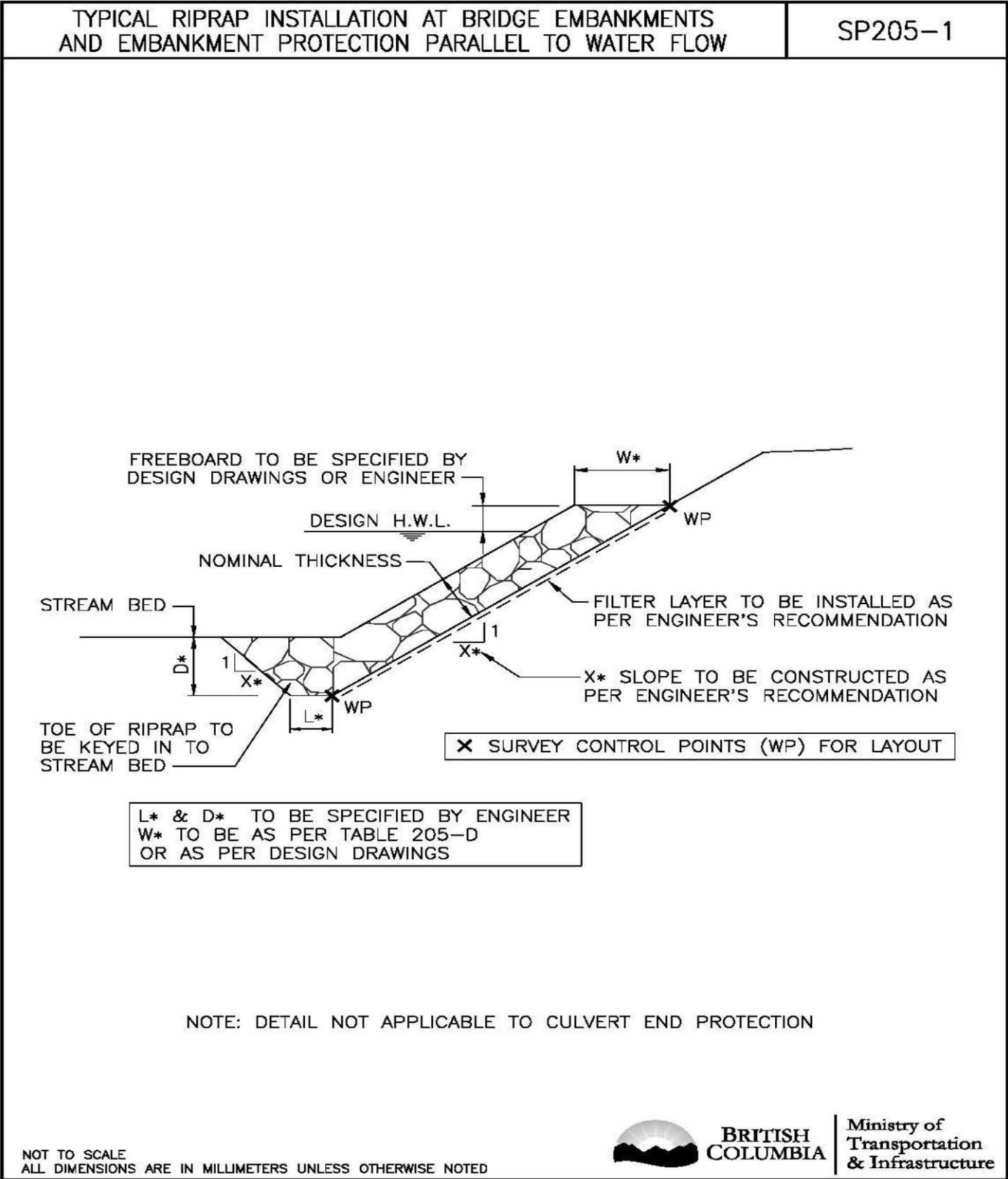
PAYMENT

205.12 Payment – Payment shall be made at the Unit Price bid per cubic metre for the Class of riprap specified or required. The Unit Price bid shall be accepted as full compensation for everything completely furnished and done in connection therewith, including supply, haul, placing, and quality control.

Payment for the excavation work at the placement site shall be paid for under "Roadway Drainage and Excavation" or "Foundation Excavation", whichever is specified in the Contract.

Where the source of supply for the riprap is off-Site or outside the design excavation limits as shown on the Drawings, the Unit Price for Riprap shall also include all costs to develop the source and produce the riprap.

Where the source of supply is on-Site and within the design excavation limits as shown on the Drawings, or within any slip, payment to excavate and haul the rock for riprap will be included within the Contract Unit Price for Type A, as defined in SS 201.



APPENDIX B

CHESTERFIELD INLET, NU, HYDROTECHNICAL ANALYSIS, PROGRESS OF WORK TO JUNE 07, 2024

1.0 INTRODUCTION

Tetra Tech Canada Inc. (Tetra Tech) is pleased to provide this hydrotechnical analysis design update memo to the Hamlet of Chesterfield Inlet regarding the Oceans Protection Plan (OPP) project to upgrade the Community's existing sealift facility.

The purpose of this memo is to:

- Summarize work complete to date.
- Outline next steps on the project and Community input required to progress the design.
- Provide estimated material quantities based on pad level at +4.0m and +3.5m.

The following feedback is required from the Community to enable Tetra Tech to progress the design:

- Define the risk level acceptable to the community, which is called 'encounter probability' of the loading condition, which may be estimated based on the return period of the design waves and the expected lifetime of the shoreline works.

If desired by the Community, Tetra Tech is available to meet virtually to discuss the contents of this technical memo and the next steps on the project.

2.0 BACKGROUND / ISSUES WITH EXISTING FACILITY

Tetra Tech Canada was retained to develop the upgrade to sealift operations at Chesterfield Inlet, NU, including the following improvements:

- Expanding the sealift laydown area via land reclamation and connecting to the Community's Road network.
- Reconstruction / resurfacing of the existing sealift ramp.
- Create a new expanded laydown area, including fill and shoreline protection.
- Clearing of boulders within the intertidal zone of the sealift ramp, to allow barge access at low tide.

Figure 1 presents the location of the proposed improvements.



Figure 1: Project Location

3.0 HYDROTECHNICAL ANALYSIS, PROGRESS OF WORK

3.1 Wind and Waves

Tetra Tech estimated the design incident wave conditions at the barge landing site using SWAN (Simulation of WAVes Nearshore; Booij et al., 1999). SWAN is a third-generation open-source wave model developed at Delft University of Technology (TU Delft). SWAN utilizes a finite difference scheme to compute random short-crested wind-generated waves and incorporates physical processes such as wave generation by wind, wave propagation, white capping, shoaling, wave breaking, bottom friction, wave refraction, wave diffraction, and non-linear wave-wave interactions in its computations.

To select the offshore boundary conditions for SWAN wave model, the Arctic Ocean Wave Hindcast data is used. This dataset is provided by the ECMWF (European Centre for Medium-Range Weather Forecasts) and is based on the output variables of the WAM model at 3 km resolution forced with surface winds and boundary wave spectra from the ECMWF ERA5 reanalysis together with ice from the ARC MFC reanalysis (Sea Ice concentration and thickness). From the output variables, significant wave height, peak period and mean direction data were extracted on the area around the SWAN grid specified by blue area in the Figure 2.

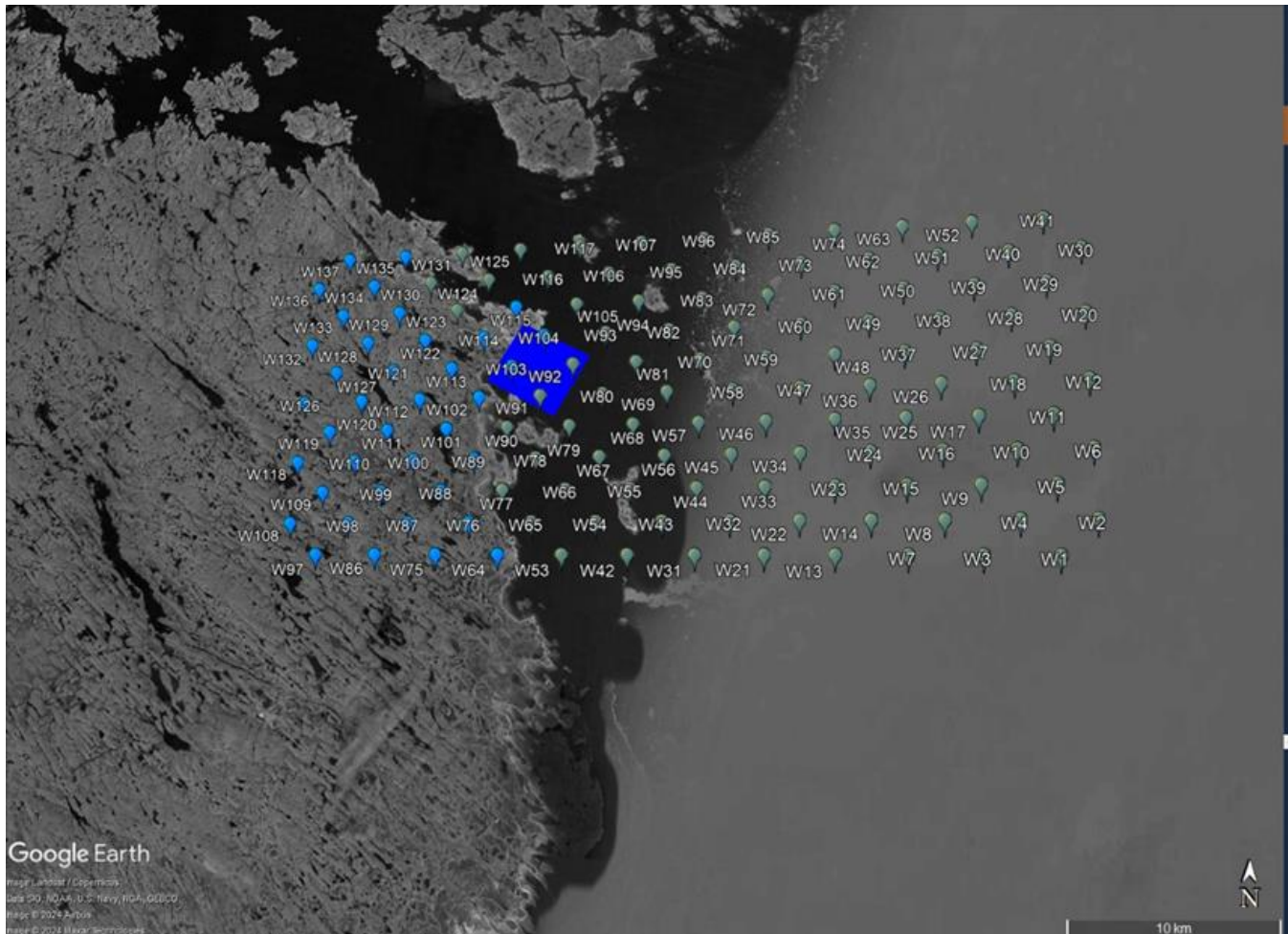


Figure 2: Wave Analysis Area

The hindcast data covers 15 years, 2008-2022, with each year includes July-November at points W91 and W92 which represent the closets WAM grid points to the offshore boundary of the SWAN grid (shown in dark blue).

The total of 146 storms with 1.5m wave height threshold ranked offshore the study area in Chesterfield Inlet. The ranking is based on the maximum significant wave height during each identified storm, defining the simulations scenario for SWAN wave model.

3.2 Tides

Tides at Chesterfield Inlet are obtained from the Canadian Hydrographic Service (CHS) chart number CHS 5620, as presented below. The chart gives depths in metres and reduced to Chart Datum (CD; Lowest Normal Tide), which at Chesterfield Inlet is 2.5 metres below Mean Water Level (MWL). The tide elevations are:

MWL	+2.5 m CD	Mean Water Level
HHWLT	+4.7 m CD	Highest High Water Large Tides
LLWLT	+0.2 m CD	Lowest Low Water Large Tides
HHWMT	+4.0 m CD	Highest High Water Mean Tides
LLWMT	+0.9 m CD	Highest High Water Large Tides

3.3 Ice Analysis

Ice analysis at Chesterfield Inlet was carried out to estimate size of shoreline protection due to ice action. It is understood that no local observations of ice thickness, or ice movements are available. Also, according to local knowledgeable, the boulders that are scattered in the intertidal zone were displaced by ice action, not wave action. Unless there would be local observations of significant ice movement toward the shoreline, the shear forces were neglected, thus riprap sizing may be based only on ice effort caused by snatching.

Based on observations of ice dynamics, ice forms in November, remains in place through the winter, and then retreats during break-up in late June-July. The frequency analysis performed was based on historical ice thicknesses, measured in Chesterfield Inlet Bay from 1960 to 1980. Considering that the ice thickness measured data were obtained offshore from the project site (< 1 km), these measurements may be conservative, as locals have mentioned that the ice breaks and is on the ground at low tide.

Based on freezing degree-days and historical ice thickness measurements, ice thickness values of 1.2 m to 1.8 m are estimated for a two-year return period (RP). Given that the ice is on the ground at low tide, the accumulated freezing degree-days over the winter do not fully contribute to ice growth, so the ice growth rate could be reduced. However, local ice thickness measurements would be needed to determine the specific growth coefficient.

The estimated D50 values, which corresponds to the average diameter (sphere size) of the stones that would be required to counter tearing from the ice, were 1.2 m to 1.6 m for the 2-year RP and 1.2-1.7 m for RP = 100-year. The return period has little effect on the D50 estimate. The estimated average diameter is Ds50 (sphere size). In coastal engineering the median characteristic diameter is used, Dn50. The relationship is $Ds50 = 1.24 Dn50$.

Thus, to resist against ice action, a Dn50 of 1 m is recommended, for a RP = 2 years, with similar values of Dn50 for RPs of up to 100 years.

3.4 Design Waves

The nearshore design waves and return periods are given as input to riprap armour design.

Hs = significant wave height (m)

Tp = peak period (s)

RP = Return Period (in years)

The significant wave height (Hs, in metres) is the average of the highest 1/3 of the waves in a wave time series. The peak period (Tp, in seconds) corresponds to the largest value from a wave spectrum (maximum energy).

The Return Period (in years) is the inverse of the probability that a given event will occur in any one year.

Design Life: Selection of the year 2050 for climate change scenarios considered a 30-year design life of the proposed laydown area fill and shoreline protection.

Table 1 presents the design wave heights for several return periods.

Table 1. Design Wave Heights

RP (years)	Hs (m)
1	1.05
2	1.2
10	1.3
20	1.37
50	1.45 (≈1.5)
100	1.5
200	1.55

3.5 Climate Change and Sea Level Rise (SLR)

Shoreline protection was designed using climate change projections for 2050, specifically the upper limit (95th percentile) of the highest emissions scenario (RCP8.5) from the IPCC's (2022) sixth assessment and including relative sea level rise (James et al., 2015), with an ice-free season leading to nearshore waves after numerical modelling transformations from the deep-water wave climate.

3.6 Design Flood Construction Levels

Flood Construction Levels (FCL) for Chesterfield Inlet laydown area and riprap protection were estimated for a suggested timeline for risk assessment of 50 years, similar than for the North Coast of BC, from Ausenco-Sandwell (2011a). General risk category was considered as 'low-moderate.'

Flood Construction Level (FCL) includes allowances for future SLR, maximum high tide (HHWLT), plus storm surge estimate, estimated wave effects, and freeboard. Table 2 gives the FCL estimates for different return periods.

Table 2. Flood Construction Level (FCL) Estimates

	Return Period RP = 2 years	Return Period RP = 10 years	Return Period RP = 50 years
Nearshore Design Wave Climate	Hs = 1.2 m Tp = 8 s	Hs = 1.3 m Tp = 8.4 s	Hs = 1.5 m Tp = 9.2 s
2050 Future Sea Level Rise Allowance (A)	0.25 m	0.25 m	0.25 m
Regional Adjustment for Uplift (B)	-0.05 m	-0.05 m	-0.05 m
High Tide (HHWLT)	4.7 m CD	4.7 m CD	4.7 m CD
Storm Surge Allowance (C)	0.3 m	0.35 m	0.4 m
Wave Effect Allowance (D)	0.1 m	0.3 m	0.85 m

Wave Set-up Allowance	0.1 m	0.1 m	0.1 m
20-years Post Construction Settlement (E)	-	-	-
Freeboard Allowance (F)	0.6 m	0.6 m	0.6 m
Flood Construction Level (FCL)	+6 m CD	+6.25 m ≈ +6.5 m CD	+6.9 m ≈ +7 m CD
FCL (Geodetic Datum)	+3.5 m GD	+4 m GD	+4.5 m GD

Notes:

(A) 2050 SLR was presented in FHALUM (2018) and Ausenco-Sandwell (2011a). The Ministry guidelines say 1 m rise by 2100, and 0.5 m rise by 2050, according to Figure 1; however, this allowance of 0.5 m of SLR (by 2050) should be adjusted plus or minus due to regional effects (e.g., crustal uplift or subsidence).

(B) This is regional adjustment could be positive or negative. With values from James et al., 2015, pages 54 and 55, there is uplift, thus a negative value.

(C) Estimated using Kamphuis (2010) storm surge equation due to wind forcing.

(D) Wave effect was calculated as 50% of wave runup estimate on the future shoreline, assuming riprap with front slope of 1(V):2(H).

(E) Post construction settlement – has not been included to date. To be defined by geotechnical analyses.

(F) The freeboard value used is based on the Ministry's Combined Method FHALUM (2018) and as recommended in Ausenco-Sandwell (2011b). Freeboard is 0.6 m or the crest elevation of an equivalent sea dike, required to keep wave overtopping below the acceptable rates.

4.0 SHORE PROTECTION

4.1 Riprap Design for Laydown Area Protection

The riprap design and dimensions follow the tables of BC MoTI (2020) Section 205, with specifications for riprap requirements (size, quality, gradations).

MoTI (2020) provides the relationship between riprap mass M (kg) and size D (m), where the diameter of the equivalent sphere (D_s) is used. However, D_s is rarely used for coastal engineering designs as is described in the Rock Manual (2007), page 107 (Chapter 3).

The size of rock is characterized by the dimension of the equivalent cube, known as the nominal diameter (D_n). For rock of density ρ_r (kg/m³), relationships between mass and size are as follows:

$$D_{n50} = (M/\rho_r)^{1/3} \text{ and}$$

$$Ds50 = 1.24 \cdot (M/pr)^{1/3}$$

For example, using $pr = 2600 \text{ kg/m}^3$, Table 205-C (MoTI, 2020, Section 205) gives the approximate values of M , and Ds , so we may adjust our technical specs to include the nominal diameter D_n (used for coastal engineering design).

Contractors normally use Ds rather than D_n . We may use Ds in the specifications, but we need to define the diameter used and be aware what it is the D_n that governs the riprap design.

The potential impact of sea ice on the riprap design was considered above. Thus, the size and shape of the riprap armour is governed by ice forces rather than wave forces.

For the typical cross-section of riprap protection, we have the following parameters:

$$D_{n50} = 1 \text{ m}$$

$$Ds50 = 1.24 \text{ m}$$

$$\text{Armour protection layer thickness: } t_a = 2.1 \text{ m}$$

Sub-layer formed by Boulders, typical sizes of 0.4 m to 0.8 m (to be removed from the nearshore and intertidal zone and saved for re-use in the riprap sub-layer). Sub-layer thickness: $t_b = 1.5 \text{ m}$

Crest elevation: equivalent to the Flood Construction Level (FCL). Three scenarios are included in Table 2 (above), to cover return periods from 2-years, 10-years, and 50-years.

Toe protection: formed by Armour stones with a width of approximately 3 m and with about 1 m of excavation, following the recommendation of MoTI (2020), as shown in Section 205, included in Appendix 1.

4.2 Elevation Design for Laydown Fill Area

Three scenarios for the laydown area pad (fill) elevation were considered, with respect to wave action on the shoreline protection riprap, varying the return periods from $RP = 2$ years to $RP = 50$ years.

The bathymetry data in drawing number L-399A (from Underhill Geomatics Ltd) dated Sep 15, 2021, shows that the buildings that are in the higher grounds, between the laydown area and the existing sealift ramp, are located in elevations between +3.5 m GD and +4 m GD.

Our hydrotechnical analysis presents three scenarios for the Laydown area pad (fill) elevation as a result of climate change on SLR, depending on the return period selected for the design waves at the site. These elevations may be referred to the Geodetic Datum (GD) or to the Chart Datum (CD) as follows:

Return Period (years)	Laydown Area Elevation/FCL (GD)	FCL (CD)
RP = 2 years	+3.5 m GD	+6 m CD
RP = 10 years	+4 m GD	+6.5 m CD
RP = 50 years	+4.5 m GD	+7 m CD

The Community (the Owner) should define the risk level they consider acceptable, which is called ‘encounter probability’ of the loading condition, which may be estimated based on the return period of the design waves and the expected lifetime of the shoreline works.

For example:

Assuming that the Lifetime of the Structure is 30 years (from 2020 to 2050), as used to calculate SLR (Table 2), we have:

Return Period (years)	Hs (Table 1)	Encounter probability of Hs exceeding the design value.
RP = 2 years	1.2 m	100%
RP = 10 years	1.3 m	96%
RP = 50 years	1.5 m	45%

Thus, for a fill reclamation with riprap protection designed for 30 years lifetime, there will be 45% of chance for the design wave height with RP = 50 years to occur (Hs = 1.5 m); and the reclamation works will probably experience Hs = 1.3 m (96% of chance) during its lifetime, with 100% of chance of the occurrence of a wave with Hs = 1.2 m.

In conclusion, note that the three options (scenarios) for the laydown area pad (fill) elevation, for a lifetime of 30 years, as considered above are:

Option 1	Pad El. +3.5 m GD	Hs = 1.2 m RP = 2 years (100% chance)
Option 2	Pad El. +4 m GD	Hs = 1.3 m RP = 10 years (96% chance)
Option 3	Pad El. +4.5 m GD	Hs = 1.5 m RP = 50 years (45% chance)

4.3 Design FCLs and SLR

If the influence of climate change and sea level rise (SLR) is not considered, the Laydown Area crest elevations would be decreased for the three options above. In this case, Table 2 rows A and B would not play a role in estimating FCLs.

Thus, the three options (scenarios) for the Laydown Area pad (fill) elevation, i.e., FCL elevations without considering the impact of climate change nor SLR, for a lifetime of 30 years for the Laydown Area, will decrease somewhat as shown below:

Scenarios without Climate Change:

Option 1 (no SLR)	Pad El. +3.3 m GD = +5.8 m CD	Hs = 1.2 m RP = 2 years (100% chance)
Option 2 (no SLR)	Pad El. +3.5 m GD = +6 m CD	Hs = 1.3 m RP = 10 years (96% chance)
Option 3 (no SLR)	Pad El. +4.2 m GD = +6.7 m CD	Hs = 1.5 m RP = 50 years (45% chance)

