SITE C HYDRO PROJECT
HIGHWAY NO. 29 REALIGNMENT AND ASSOCIATED ROADS

HYDRAULIC DESIGN REPORT
FARRELL CREEK BRIDGE

100% SUBMISSION – FINAL REPORT
REVISION 1

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Disclaimer

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CREDITS AND ACKNOWLEDGEMENTS

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EXECUTIVE SUMMARY

BC Hydro is constructing the Site C Dam on the Peace River and the future reservoir will inundate the portions of Highway 29 that cross the lower reaches of five major tributaries to the Peace River: (i) Lynx Creek, (ii) Dry Creek, (iii) Farrell Creek (iv) Halfway River and (v) Cache Creek and a portion of the Highway that runs adjacent to the Peace River between Lynx Creek and Dry Creek known as “Lynx East” (Figure 1-1). The affected segments of Highway 29 will be realignment northward and will cross the tributary embayments with larger structures than exist today.

Northwest Hydraulic Consultants Ltd. (NHC) has been retained as part of the multidisciplinary, design team led by R.F. Binnie and Associates (Binnie) to provide hydraulic design input for new bridges and highway embankments in each segment.

Farrell Creek drains an area of 642 km² upstream of the proposed Highway 29 Bridge. At present the main channel is 20-25 m wide, measured at the top of bank. The channel gradient is between 0.7 % and 0.8 %. The bed material near the new bridge crossing consists of predominantly gravels and cobble, mixed with sand and silt. The main channel encroaches upon the right valley wall at the new 6-span 411 m long bridge crossing, a near-vertical 30 m high slope of fractured, bedrock with overlying gravel. A 150 m wide floodplain extends east from the left bank towards the toe of the left valley wall. The left valley wall consists of a densely vegetated lower bank that extends up to the existing Highway 29 alignment. A sparsely vegetated upper bank comprised of erodible granular material then extends up to the crest. A compacted granular berm will be constructed at the toe of the left valley wall and below Pier 4 to reduce the required lengths of the pier columns and provide access for construction.

The climate station at Fort St. John (Station #1183000) provides a record of climate data: daily air temperature, precipitation, snow on ground, bright sunshine, and wind. Daily average temperatures range from a low of -12.8 degrees (°) Celsius in January to a high of 16 ° Celsius in July. Sub-zero temperatures prevail from November through to the end of March and the typical average winter temperature is -9.0 ° Celsius. The minimum solar radiation occurs in December with a daily average of 1.90 megajoules per square metre (MJ/ m²/ day) per day. This is equivalent to an air temperature offset of about 1 degree Celsius. In March, the solar radiation increases to 11.7 MJ/ m²/ day, which is equivalent to about 7 ° Celsius in additional air temperature. In terms of energy budget, solar radiation does not play a significant role in the winter, but it is very important in the spring.

Fort St. John receives just over 290 mm of rainfall per year, with more than 90 percent of it occurring between April and September (270 mm). About 190 cm of snow fall each year, mostly between October and April (179 cm). The total annual precipitation (water equivalent) amounts to 445 mm per year.

An assessment of climate change impacts on temperature and rainfall has been undertaken using PCIC’s Plan2Adapt Tool. Projected increases in daily temperature range from +1.7 °C to +4.5 °C, with a median value of 2.8 °C. Projected increases in daily rainfall range from -10% to +17% in the 2080s. The median projected increase is +1%.
To support the Site C Project, BC Hydro has been recording wind data within the Peace River valley since 2010 using project specific meteorological stations. 200-year valley wind speed estimates at Farrell Creek has been estimated from hindcasted records at the BC Hydro stations, using the historical record at Fort St John as a baseline. The 200-year valley wind speeds for each direction are: NE 12.0 m/s; SE 11.2 m/s; SW 20.9 m/s; and NW 11.5 m/s. The estimates are used in the design of erosion protection at the bridge.

The hydrology for Farrell Creek has been assessed using regional streamflow data collected by the Water Survey of Canada (WSC). The peak design flow during the construction period in Farrell Creek is the 10-yr peak flow, estimated as 155.0 m³/s. Based on a recent climate change impact assessment for the Peace Region (NHC, 2017), it is plausible that the 200-yr peak inflow to Farrell Creek could increase by 20% by the end of the 21st Century. However, once the Site C Reservoir is created, water flowing within the embayment beneath the bridge will be deep with very low velocity, even during large floods. Potential increases in flood flow due to climate change will result in no change in depth and only negligible change in velocity, and thus will have no bearing on the design of the bridge.

A detailed assessment of the ice regime and effects of ice on the bridge has been carried out. The 50-yr ice thickness is recommended for use in the estimation of ice loads. The maximum 50-yr thickness is estimated to be 0.73 m, and the 50-yr spring ice thickness is estimated to be 0.67 m. For ice strength, a mid-winter flexural strength of 1,000 kPa is recommended while a spring crushing ice strength of 700 kPa and a spring flexural strength of 590 kPa. For the situation where large spring floes interact with a shoreline over much larger scales – for example, the diameter of the floe – it is appropriate to adopt a lower crushing strength of 400 kPa. In this case, the corresponding flexural strength would be 340 kPa.

A two-dimensional numerical model was developed to assess hydraulics at the Farrell Creek crossing during the construction period. Two model scenarios were run in the model. A summary of the model results is summarized in the table below:

<table>
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<th>Scenario</th>
<th>Description of Farrell Creek Flow</th>
<th>Flow (m³/s)</th>
<th>WSEa (m)</th>
<th>Average Velocity (m/s)</th>
<th>Maximum flow depth (m)</th>
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</thead>
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<td>1</td>
<td>10-yr peak instantaneous</td>
<td>155</td>
<td>443.2</td>
<td>3.4</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>Mean Annual Flood</td>
<td>77</td>
<td>442.8</td>
<td>2.2</td>
<td>1.6</td>
</tr>
</tbody>
</table>

Wind waves at the segments have been assessed by means of two dimensional numerical analysis using the SWAN wave model, discussed in Section 6. 200-yr wind wave heights within the Farrell Creek embayment are predicted to range between 0.3 m and 1.1 m. The maximum wave height produced by recreational vessels is estimated at 1 m.
Geotechnical studies conducted by BGC (2012) found that a hypothetical landslide with a return period of 10,000 years occurring on Peace River south bank opposite to the Farrell Creek confluence could produce landslide generate waves (LGWs) that could impact the proposed bridge structure and approach fills. Flow3D software was used to analyse impact of a 10,000-yr LGW and a 1,000-yr LGW to the new bridge structure and the approach fills. Details of the latest LGW modelling can be found in the technical memorandum in Appendix C.

Pier 1 berm and Pier 2 may be exposed to flowing water during construction and require protection. At the Pier 1 berm riprap will tie into high ground upstream of the transition from the existing west valley wall to Pier 1 berm, around the berm and extending downstream a short distance along the existing west valley wall. The Pier 2 isolation berm will consist of a riprap lined berm (with 2H:1V side slopes, crest width of 2 m and elevation minimum 0.3 m above the 10-year flow) that surrounds the Pier 2 construction area and ties into the east floodplain. The design is detailed on drawings 2184-281 and 2184-283 (Appendix B).

In order to ensure construction of Pier 1 and Pier 2 occur in the dry, Farrell Creek must be realigned and confined to a single channel east of its present location. Responsibility for the realigned channel will fall to the contractor, however, a suggested realigned channel is detailed on the T3 Reference drawing (Appendix B).

Scour under construction conditions has been evaluated. The potential for contraction scour is low as the proposed bridge does not constrict the realigned main channel. For Pier 1 berm and the Pier 2 isolation berm the 10-yr natural scour depth is 3.0 m below the 10-yr flood elevation, or El. 440.2 m. Riprap along the Pier 1 berm and Pier 2 isolation berm will be keyed in below the natural scour condition. There will be natural scour potential along the toe of riprap placed in the realigned channel. The Contractor will be responsible for estimating and designing for such scour.

A temporary bridge is required to provide access to a gravel source over Farrell Creek upstream of the existing Highway 29 bridge. The temporary bridge soffit must be 0.5 m above the predicted 10-yr flood elevation. The bridge opening must have a bottom width of 20 m with 2H:1V side slopes.

The 1,000- and 10,000-yr LGWs are not expected to impact the bridge deck, girders, or abutments. The maximum overall horizontal pressure force occurs at Pier 2 at approximately 1,839 kN for 10,000-yr LGW.

Dynamic ice loads on the conical portion of the pier columns are estimated to vary between 1.90 and 2.26 MN. Given that the maximum ice level is very close to the top of the cone, the ice could ride up and fail in crushing against the 2.4 m diameter vertical pier. This force would act at El. 462.6 m. The full crushing force associated with a floe diameter of 146 m would be 1.74 MN. The bottom of the cone is about 0.2 m above the lower limit of the normal ice action of 459.3 m; a crushing force of 0.56 MN could develop on the 3.5 m diameter pier at El. 459.4 m. During a drawdown event the ice level may drop below the level of the cone so dynamic ice forces may occur on the 3.5 m diameter portion of the pier. A 10-yr spring ice thickness of 0.57 m and a 10-yr terminal floe velocity of 0.77 m/s is recommended for
this type of event because drawdown is unlikely to occur more than once during the life of the structure. The corresponding energy limited crushing force for the 3.5 m diameter pier is 1.79 MN.

The maximum uplift and downdrag forces for the piers would occur when the water level is at El. 460.0 m and the corresponding diameter of the cone would be 3.43 m. The design force for this situation would be 1.03 MN. During a drawdown event the ice level may drop below the level of the cone so uplift and downdrag forces may occur on the 3.5 m diameter portion of the pier. A 10-yr mid-winter ice thickness of 0.63 m is recommended for this type of event. The corresponding uplift and downdrag force is 0.80 MN.

In the embayment, the largest thermal force will potentially occur in winter when the ice is at its maximum thickness and there is a large increase in air temperature. The worst-case force acting in any direction would be 0.78 MN. These forces are less than the maximum flexural forces that the ice sheet can sustain and thus they are not limited by the flexural strength of the ice sheet. During a drawdown event the ice level may drop below the level of the cone so thermal forces may occur on the 3.5 m diameter portion of the pier. A 10-yr mid-winter ice thickness of 0.63 m is recommended for this type of event. The corresponding thermal force is 0.61 MN. Ice forces on riprap are not significant and do not govern the design of riprap.

A discussion of erosion protection is presented in Section 7.6. Gradations for protection against stream erosion and wind/boat waves will conform to BCMOT Riprap Classes.

During normal operation of the reservoir, erosion of the side slope adjacent to Pier 5 is of concern. The side slopes of the Farrell Creek valley are predominantly weathered bedrock within the wave erosion zone. Erosion of the bedrock from wave and ice action is of particular concern along the east valley slope at Pier 5 as the east abutment encroaches upon the Stability Impact Line (SIL), as per hydrotechnical design criteria erosion protection is required for this scenario. The design return period for erosion protection against wind and boat waves, and ice forces is 200 years. The design wave is the wind wave, which has a height of 1.1 m, and the required D50 of riprap to protect against this wave corresponds to BCMOT Class 250-kg riprap or MCW Zone 6e riprap. For layout and extents of the riprap placement see drawing 2184-282 and 2184-283 (Appendix B).

On rare occasions it may be necessary to draw down the reservoir in order to carry out maintenance or repair activities, which will expose the previously inundated Pier 1 granular berm slopes to erosion by wave erosion and possibly stream currents. A design return period of 5 years is appropriate since the duration would only be for upwards of 6 months. The minimum expected drawdown is to El. 444.0.

During construction Pier 1 berm and Pier 2 isolation berm will be exposed to flowing water and will require MoTI Class 250-kg riprap to protect against a 10-yr return period flood. The Contractor will be responsible for estimating and design of the riprap within the realigned channel. For layout and extents of the riprap placement see drawing 2184-281 and 2184-283 (Appendix B).
LIST OF ACRONYMS

BOR           The bottom elevation of riprap at a given location
BCMOT         British Columbia Ministry of Transportation and Infrastructure
EI.           Elevation above mean sea level in Geological Survey of Canada (GSC) datum
LGW           Landslide generated wave
MAF           Mean annual flow: the average flow in a stream in a year
NHC           Northwest Hydraulic Consultants Ltd. (author of this report)
S6-14         Canadian Standards Association Highway Bridge Design Code
TOR           The top elevation of riprap at a given location
WSC           Water Survey of Canada
WSE           Water surface elevation

GLOSSARY

Left and Right When used in reference to stream channels, these assume the subject is looking downstream
Mean Annual Flood Arithmetic mean of annual maximum flows over the period of record.
Mean Annual Flow  Arithmetic mean of daily average flow over the period of record
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Appendix C Farrell Creek 1,000 and 10,000-yr LGW Impacts Memo (R2)
1.0 INTRODUCTION

BC Hydro is constructing the Site C Dam on the Peace River and the future reservoir will inundate the portions of Highway 29 that cross the lower reaches of five major tributaries to the Peace River: (i) Lynx Creek, (ii) Dry Creek, (iii) Farrell Creek (iv) Halfway River and (v) Cache Creek and a portion of the Highway that runs adjacent to the Peace River between Lynx Creek and Dry Creek known as “Lynx East” (Figure 1-1). The affected segments of Highway 29 will be realigned northward and will cross the tributary embayments with larger structures than exist today.

Northwest Hydraulic Consultants Ltd. (NHC) has been retained as part of the multidisciplinary design team led by R.F. Binnie and Associates (Binnie) to provide hydraulic design input for new bridges and highway embankments in each segment.

This report presents the 100% hydraulic detailed design for the new 411m long, six-span bridge at Farrell Creek. The bridge will be located roughly 150 m south of the existing Highway 29 crossing.

1.1 Objectives and Organization of this Report

The objectives of this report are to: (i) provide background to the Highway 29 Realignment Project for Farrell Creek in particular; (ii) describe the current and future climate in proximity to, and the hydrology and hydraulic environment of Farrell Creek; and, (iii) summarize the hydraulic analysis and design completed by NHC in support of Binnie’s Design Team.

Section 2 of the report describes the existing river setting and the effects of the future reservoir. Sections 3 through 6 discuss climate, hydrology, the current and future ice regime and the future wave climate. Those four sections build the basis for hydraulic design, which is described in Section 7. Section 8 contains a list of references.

There are three Appendices which contain a greater level of detail on certain topics covered in the main body of the report.
FIGURE 1-1
HIGHWAY 29 RELOCATION AND ASSOCIATED ROADS
TRIBUTARY CROSSINGS

DATA SOURCES:
BACKGROUND - NATIONAL GEOGRAPHIC
INSET BACKGROUND - ESRI SHADED RELIEF

TRIBUTARY STREAMS
- WESTERN SEGMENTS CROSSING
- OTHER SITE C CROSSING

COORDINATE SYSTEM: NAD 1983 UTM ZONE 10N
UNITS: METRES

SCALE: 1:250,000
0 2 4 6 8 10 KM

BChydro
northwest hydraulic consultants

job: 3002853 date: 25-jul-2019
1.2 Hydrotechnical Design Criteria

A memorandum summarizing the hydrotechnical design criteria for the Project can be found in Appendix A.

1.3 Design Conditions

A key aspect of the hydraulic design is consideration of evolving conditions as the Project moves from construction through reservoir filling, and then to normal reservoir operations. Emergency drawdown of the reservoir is also considered.

1.3.1 Construction Conditions

During construction Farrell Creek will be flowing under normal conditions. The construction access roads, berms, floodplain and bridge piers will be subject to erosion and scour. The construction period is expected to last 1 to 2 years and the recommended design return period is 10 years.

1.3.2 Reservoir Filling Conditions

The reservoir filling period is anticipated to last for 60 to 90 days depending on the rate of inflow at the Dam (Klohn Crippen Berger, 2014). The filling rate could vary between 0.4 m/ day and 1.3 m/day, so if the filling operation is not interrupted, no areas should be exposed to wind wave erosion for more than a day. Once the reservoir starts to fill, streamflow velocities on Farrell Creek will diminish rapidly so bank erosion by stream currents is not a concern. A 5-year return period is considered appropriate for design.

1.3.3 Normal Operating Conditions

Under normal operating conditions, the reservoir level will fluctuate between maximum normal reservoir level (MNRL) at El. 461.8 m and minimum normal reservoir level (MnNRL) at El. 460.0 m. Exposed slopes will be subject to erosion from wind and boat waves, ice forces and, potentially, landslide-generated waves (LGWs). The design return period for erosion protection against wind and boat waves is 200 years. Design return periods for forces related to ice action are on the order of 1,000s of years and the design LGWs for the highway and bridge have return periods of 1,000 years and 10,000 years, respectively.

1.3.4 Drawdown Conditions

On rare occasions it may be necessary to draw down the reservoir in order to carry out maintenance or repair activities, which will expose the previously inundated slopes to erosion by wave erosion and possibly stream currents. A design return period of 5 years is appropriate for riprap protection since the duration would be a maximum of 6 months. For dynamic longitudinal ice loading on bridge piers the 10 year spring ice thickness and floe velocity is appropriate and for uplift, downdrag and thermal forces acting on bridge piers the 10 year mid-winter ice thickness is appropriate as drawdown is unlikely to
occur more than once during the lifetime of the structure. The minimum expected drawdown is to El. 444.0 (KCB, 2016).

1.4 Codes and Reference Used

The following design codes and references have been used during study:

- CSA S6-14 (S6-14)
- BC MOT Supplement to S6-14 (2016)
- Codes and standards established by Federal and Provincial agencies for protection of the environment and navigable waters
- Additional design procedures and research, such as US federal Highways Association Manuals, and others referenced in the report
2.0 RIVER SETTING

2.1 Existing Conditions

2.1.1 Peace River

Downstream of WAC Bennett Dam, Peace River is entrenched within a valley that it has cut in the high plains of the Alberta Plateau (Church 2005). The plateau is underlain by moderately to weakly lithified shale and sandstone rocks dating to the Cretaceous, while substantial sediments of Pleistocene age are found along the valley. The valley was occupied by the pro-glacial Lake Peace, creating extensive deposits of silt up to 40 m in thickness (Slaymaker 1972) that are subject to landslides that can enter the river, and have in the past, temporarily blocked the flow (e.g. the Attachie Slide of 1972).

2.1.2 Farrell Creek at the Proposed New Bridge

Farrell Creek drains an area of 642 km² upstream of the proposed, new Highway 29 Bridge. The bed material near the proposed crossing consists of predominantly gravels and cobble, mixed with sand and silt (Photo 2-1).

Photo 2-1 Farrell Creek bed material

The existing Highway 29 bridge crosses Farrell Creek 150 m upstream of the new bridge location. 500 kg Class riprap protects both abutments and extends a short distance upstream and downstream.
The channel gradient is between 0.7 % and 0.8 %. Immediately downstream of the existing Highway 29 bridge the channel bends to the left\(^1\) with flood channels located to the right and left of the main channel, separated by low, vegetated bars. The flood channels are 10-15 m wide, measured at the top of the banks, and they rejoin the main channel at the new bridge location (Photo 2-2).

![Photo 2-2](image.png) Looking downstream from the existing bridge (dashed orange lines show flood channels)

Along the centreline of the new 6-span 411 m long bridge, the main channel is 20-25 m wide, measured at the top of bank. The main channel encroaches upon the right valley wall, a near-vertical 30 m high slope of fractured, bedrock with overlying gravel. A 150 m wide floodplain extends east from the left bank of the main channel towards the toe of the left valley wall. The left valley wall consists of a densely vegetated lower bank that extends up to the existing Highway 29 surface. The sparsely vegetated upper valley wall is comprised of erodible granular material.

A layout of the proposed bridge and granular berms can be seen on drawings 2184-281 to 283 (Appendix B). The compacted granular berms at Pier 1 and Pier 4 are intended to reduce the required lengths of the pier columns and provide access for construction.

Pier 1 will be located on a granular berm that will be located along the toe of the existing steep right valley wall. Piers 2 through 4 will be located at 74 m intervals across the valley bottom and floodplain with Pier 2 located along the gravel bar left of the main channel and Pier 4 on a granular berm. Pier 5 will be located at the toe of the left valley wall where the existing Highway 29 transitions to the bedrock slope.

Downstream of the new bridge the channel is unconfined as it joins the Peace River.

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\(^1\) Left and right with respect to looking downstream
2.2 General Effects of the Proposed Future Reservoir

The embayment created by the future reservoir will have much higher water levels than the present Farrell Creek channels and near-zero flow velocities resulting in an expected increase in the size and number of recreational boats utilizing the tributary.

The winter ice regime will be far different than what exists on the tributaries today. A relatively thick ice cover is expected to form in the embayment and will impose significant loads on the piers of the new Farrell Creek bridge.

Hydraulic erosion of the tributary banks and highway fill slopes will shift from being primarily driven by stream currents to being driven by wind waves and boat waves. There is also a risk of landslide generated waves (LGWs) originating from slope failures of varying sizes and along the southern valley wall adjacent to the future reservoir (BGC 2010). Church (2005) suggests that east of the Rocky Mountains, the Peace River valley sides are prone to landslides along a continuous length of more than 800 km. Erosion from the largest LGWs would be extreme, although these events have an annual probability of occurrence in the range of 1 in 1,000 to 1 in 10,000 years (BGC, 2013). Detailed discussion of the design landslide and LGW is presented in Section 6.3.

A new delta will form at the upstream limit of the Farrell Creek embayment, which will effectively eliminate bed material transport resulting in aggradation and an increase in lateral channel shifting like what exists at the present-day delta at the Peace River. Finer sediments will continue to move in suspension and settle within the embayment resulting in some loss of storage over time.
3.0 CLIMATE AND HYDROLOGY

3.1 Climate

The climate stations at Fort St. John Airport (Station #1183000) and Hudson’s Hope BCHPA Dam (1183FL0) provide historical records of climate data such as daily air temperature, precipitation, snow on ground and bright sunshine. Fort St. John also has historical records of hourly wind data. The Hudson’s Hope records are relatively short, so the Fort St John record provides the basis for most of the discussion in the following sections. Unless otherwise noted, the statistics referred to in this section are based on climate normal for the period 1980-2010.

3.1.1 Air Temperatures and Degree-Days of Freezing

Daily average temperatures at Fort St. John range from a low of -12.8°C in January to a high of 16.2°C in July. Sub-zero temperatures prevail from November through to the end of March and the typical average winter temperature is about -9.0°C.

The average degree days of freezing over the winter was 1,415 degree-days, with a 10% chance each year that it could be less than 1,015 or greater than 1,860 degree-days. This data has been used to evaluate the expected range of ice thickness on the future reservoir.

3.1.2 Bright Sunshine

Bright sunshine data is used to estimate incoming solar radiation, an important parameter in ice growth and deterioration modelling. The record at Fort St. John is incomplete and it was necessary to use data from Beaverlodge, AB (Station 3070560), Fort Nelson, BC (Station 1192940), and Germansen Landing, AB (Station 1183090) to augment the record. Still, there were gaps that had to be filled using long term average sunshine hours. The daily bright sunshine hours at Fort St. John were converted to daily solar radiation values using the relationship between it and the Beaverlodge data after accounting for the slight difference in latitude.

The minimum solar radiation occurs in December with a daily average of 1.90 megajoules (MJ)/m²/day. This is equivalent to an air temperature offset of about 1°C. In March, the solar radiation increases to 11.7 MJ/m²/day, which is equivalent to about 7°C in additional air temperature. In terms of energy budget, solar radiation does not play a significant role in the winter, but it is very important in the spring.

3.1.3 Precipitation

Fort St. John receives just over 290 mm of rainfall per year, with more than 90 percent of it occurring between April and September (270 mm). About 190 cm of snow fall each year, mostly between October and April (179 cm). The total annual precipitation (water equivalent) amounts to 445 mm per year.
Current Intensity-Duration-Frequency (IDF) data for Fort St. John is shown in Table 3-1. These data are based on sub-daily rainfall intensity statistics through the year 2002.

Table 3-1  Summary of rainfall intensities at Fort St. John Airport (Station #1183000; 1973-02)

<table>
<thead>
<tr>
<th>Duration Unit</th>
<th>2-yr mm/hr</th>
<th>5-yr mm/hr</th>
<th>10-yr mm/hr</th>
<th>25-yr mm/hr</th>
<th>50-yr mm/hr</th>
<th>100-yr mm/hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 min</td>
<td>52.8</td>
<td>77.2</td>
<td>93.3</td>
<td>113.7</td>
<td>128.8</td>
<td>143.8</td>
</tr>
<tr>
<td>10 min</td>
<td>37.6</td>
<td>53.0</td>
<td>63.3</td>
<td>76.2</td>
<td>85.9</td>
<td>95.4</td>
</tr>
<tr>
<td>15 min</td>
<td>30.5</td>
<td>42.9</td>
<td>51.2</td>
<td>61.6</td>
<td>69.4</td>
<td>77.1</td>
</tr>
<tr>
<td>30 min</td>
<td>20.2</td>
<td>29.0</td>
<td>34.8</td>
<td>42.2</td>
<td>47.6</td>
<td>53.0</td>
</tr>
<tr>
<td>1 h</td>
<td>11.9</td>
<td>16.9</td>
<td>20.3</td>
<td>24.5</td>
<td>27.6</td>
<td>30.67</td>
</tr>
<tr>
<td>2 h</td>
<td>6.9</td>
<td>9.7</td>
<td>11.6</td>
<td>13.9</td>
<td>15.7</td>
<td>17.4</td>
</tr>
<tr>
<td>6 h</td>
<td>3.4</td>
<td>4.5</td>
<td>5.2</td>
<td>6.1</td>
<td>6.8</td>
<td>7.4</td>
</tr>
<tr>
<td>12 h</td>
<td>2.3</td>
<td>3.0</td>
<td>3.4</td>
<td>3.9</td>
<td>4.3</td>
<td>4.7</td>
</tr>
<tr>
<td>24 h</td>
<td>1.5</td>
<td>1.9</td>
<td>2.1</td>
<td>2.4</td>
<td>2.7</td>
<td>2.9</td>
</tr>
</tbody>
</table>

Snow has historically occurred in every month except July and about 190 cm falls on average each year. Monthly snowfall amounts of around 25 to 30 cm occur November through March.

3.1.4  Projected Increases in Temperature Due to Climate Change

The potential impacts of climate change on mean daily air temperature has been assessed using the Pacific Climate Impacts Consortium’s (PCIC’s) Plan2Adapt Tool. Projected increases range from +1.7°C to +4.5°C, with a median value of 2.8°C. The Plan2Adapt projections are based on outputs from an ensemble of fifteen (15) Global Climate Models (GCMs), each using a high (A2) and low (B1) greenhouse emissions scenario. The projected increases are relative to a baseline historical normals from 1961-1990, meaning the baseline period is now almost 30 years old.

3.1.5  Projected Increases in Rainfall Due to Climate Change

The potential impacts of climate change on rainfall has been assessed using both the Pacific Climate Impacts Consortium’s (PCIC’s) Plan2Adapt Tool and the web-based, IDF-CC Tool (V. 3.0; Simonovic et al, 2018).

Plan2Adapt shows daily rainfall increases in the Peace River Region ranging from -10 % to +17 % in the 2080s. The median projected increase is +1 %.

Projected increases in sub-hourly rainfall, to the end of the 21st century (2100) have been assessed using the IDF-CC Tool. It uses daily precipitation projections from nine (9) bias corrected models developed by PCIC for Environment Canada, in addition to the GCMs used with earlier versions of the software. It uses a simulation period from 1950-2100 and relates projects to the most recently published IDF data from
2002 (Table 3-1). NHC assessed the projections for the Representative Concentration Pathway (RCP) 8.5 only. IDF-CC predicts much larger increases in rainfall than does (PCIC’s) Plan2Adapt Tool – up to 70% increases in some cases. NHC’s understanding is that the IDF-CC Tool assumes a constant scaling relationship between daily and sub-daily rainfall.

3.1.6 Wind

The historical mean all-direction wind speed at Fort St. John is about 4 m/s, but maximum hourly wind speeds range from 15 to 26 m/s and gust speeds can be as high as 38 m/s. The winds are from the southwest and west-southwest for about 30 % of the time and from the north for about 14 % of the time. Northwest winds occur relatively infrequently relative to other areas in the region. The mean annual wind speed also follows the same pattern with the highest winds coming from the southwest and the west-southwest, followed by winds from the north and northwest – when they occur. The wind rose for Fort St. John Airport is shown in Figure 3-1 and average recurrence intervals of windspeed are shown in Table 3-1.

Figure 3-1  Wind rose distributions for Fort St. John Airport (1953-2018)
### Table 3-2  Return periods of wind speed for Fort St. John Airport (1953-2018)

<table>
<thead>
<tr>
<th>Return Period (yr)</th>
<th>Wind Speed (m/s) Northeasterly</th>
<th>Wind Speed (m/s) Southeasterly</th>
<th>Wind Speed (m/s) Southwesterly</th>
<th>Wind Speed (m/s) Northwesterly</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>11.46</td>
<td>11.65</td>
<td>19.05</td>
<td>15.91</td>
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<td>5</td>
<td>12.51</td>
<td>12.88</td>
<td>20.65</td>
<td>17.15</td>
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<tr>
<td>10</td>
<td>13.23</td>
<td>13.73</td>
<td>21.77</td>
<td>18.01</td>
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<tr>
<td>20</td>
<td>13.93</td>
<td>14.55</td>
<td>22.86</td>
<td>18.84</td>
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<tr>
<td>50</td>
<td>14.83</td>
<td>15.62</td>
<td>24.27</td>
<td>19.93</td>
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<tr>
<td>100</td>
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<td>16.42</td>
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<td>200</td>
<td>16.19</td>
<td>17.22</td>
<td>26.40</td>
<td>21.56</td>
</tr>
</tbody>
</table>

An analysis of winds within the current Peace River Valley, where the main body of the Site C Reservoir will be located, was performed using hindcasted wind data. Records of five to seven years duration of wind data at seven BC Hydro-owned meteorological stations within the reservoir footprint were acquired (Figure 3-2). Observations from each of these stations were correlated to wind observations at Fort St. John Airport to hindcast a long-term (1953-2017) wind record at each BC Hydro Station. The hindcasted, long-term wind records were used for frequency analysis and to generate maximum hourly wind speeds of various average recurrence intervals (ARI). An in-depth discussion of the hindcast procedure can be found in the Functional Design Hydraulics Report for Halfway River (NHC, 2018a).

To be conservative, the maximum hourly wind speeds for the Peace Valley are taken as the strongest winds at any of the stations within a particular sector (Table 3-3).

![Figure 3-2 Location of BC Hydro wind stations along the Peace Valley and future Site C Reservoir](image-url)
### Table 3-3 Maximum hourly wind speeds, Peace Valley within the main body of the Site C Reservoir

<table>
<thead>
<tr>
<th>Location</th>
<th>Sector</th>
<th>ARIa</th>
<th>Wind Speed</th>
<th>Reference BC Hydro Stationb</th>
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<tbody>
<tr>
<td></td>
<td>NE</td>
<td>2</td>
<td>8.8</td>
<td>Station 3</td>
</tr>
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<td></td>
<td></td>
<td>5</td>
<td>9.5</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>10</td>
<td>10.0</td>
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<td>10.5</td>
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<td>50</td>
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<td></td>
<td></td>
<td>200</td>
<td>12.0</td>
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<td>2</td>
<td>7.4</td>
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<td>2</td>
<td>15.3</td>
<td>Station 1</td>
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<td>200</td>
<td>11.5</td>
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</tbody>
</table>

**Notes**

a Average recurrence interval  
b See Figure 3-2 for the location of BC Hydro Stations

In addition to Peace River Valley winds in general, maximum hourly wind speeds and directions are determined for all the Western Segments.

Wind speeds were selected based on the return period winds established for all project sites (Table 3-3) and assumed to blow at each of the BC Hydro stations considered to define the Peace River Valley wind field for a simulation. Wind direction was set specific to each site, inline with local fetches, in order to produce most adverse wave conditions.

Because of uncertainties regarding the exposure of some BC Hydro stations, notably those located only a few hundred metres away from forests and nearby hills, maximum hourly wind speeds for a given wind sector are considered the same for all Western Segment sites and are established according to the
strongest winds at any of the BC Hydro stations for that sector. A summary of the predicted maximum hourly windspeeds at the Western Segments are presented in Table 3-4.

**Table 3-4  Predicted maximum hourly windspeed estimates at Farrell Creek**

<table>
<thead>
<tr>
<th>Wind Simulation ID</th>
<th>General Wind Direction Sector</th>
<th>ARI</th>
<th>Reference BC Hydro Stations</th>
<th>Station 5</th>
<th>Station 6</th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>WS (m/s)</td>
<td>Dir (°)</td>
</tr>
<tr>
<td>FC-Prod-E-100_002</td>
<td>E</td>
<td>100</td>
<td></td>
<td>11.6</td>
<td>90</td>
</tr>
<tr>
<td>FC-Prod-E-200_002</td>
<td>E</td>
<td>200</td>
<td></td>
<td>12.0</td>
<td>90</td>
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<td>FC-Prod-SSE-200_002</td>
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<td></td>
<td>11.2</td>
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<td>180</td>
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<td>25</td>
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</tbody>
</table>
3.2 Hydrology

3.2.1 Peace River Tributaries

Hydrologic estimates for Farrell Creek have been determined using regional streamflow data collected by the Water Survey of Canada (WSC). WSC 07FB005 Blueberry River was judged to be the best predictor for Farrell. Table 3-5 presents details of the gauge; regional watershed base map with Farrell Creek highlighted is presented in Figure 3-3. Estimates of mean annual and mean monthly flow were determined for Blueberry River WSC record, and frequency analyses of Blueberry River WSC annual maximum daily and annual peak flow data were used to estimate mean annual floods and peak flow quantiles for return periods ranging from 2 to 200 years.

Mean annual and mean monthly flow estimates for Farrell Creek were then made by linear drainage area scaling. Annual peak flow return periods were scaled to Farrell Creek with an exponent$^2$ of 0.75 using Equation 3-2:

\[
P_{k, \text{flow} \text{ crossing}} = P_{k, \text{flow} \text{ WSC gauge}} \times \left(\frac{\text{drainage area} \text{ crossing}}{\text{drainage area} \text{ WSC gauge}}\right)^{0.75} \quad (\text{Eq. 3-2})
\]

Results for the mean monthly and mean annual flow estimates are presented in Table 3-6. Annual maximum daily and annual peak flow estimates are presented in Table 3-7.

---

$^2$ Exponent is a regional value
### Table 3-5  Basin characteristics for Blueberry Creek WSC station and Farrell Creek

<table>
<thead>
<tr>
<th>Drainage Area</th>
<th>WSC ID</th>
<th>Drainage Area (km²) a</th>
<th>Mean Elev. (m) b</th>
<th>Elevation Range (m) b</th>
<th>Lake (%) c</th>
<th>Aspect a</th>
<th>Hydrologic Zone d</th>
<th>Period of Record</th>
<th>Annual Peaks (yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WSC Gauge</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Blueberry River below Aitken Creek</td>
<td>07FC003</td>
<td>1773</td>
<td>826</td>
<td>648-1,047</td>
<td>0.05</td>
<td>SE</td>
<td>6</td>
<td>1965-2010</td>
<td>44</td>
</tr>
<tr>
<td>Peace River Tributary Crossings at Highway 29</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Farrell Creek</td>
<td></td>
<td>642</td>
<td>783</td>
<td>449-1,693</td>
<td>0.3</td>
<td>SE</td>
<td>3, 6, 7</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:

a Drainage areas and aspect derived from BC 1:20,000 scale Freshwater Atlas (FWA), 1:50,000 scale Canadian Digital Elevation Data (CDED).
b Elevations based on CDED.
c Glacier and lake coverage determined from FWA.
d Zones: 3. Northern Rocky Mountains, 6. Southern Interior Plains, 7. Southern Rocky Mountain Foothills; descriptions of these zones are provided in Obedkoff (2003) and references therein.
### Table 3-6  Mean monthly flow estimates

<table>
<thead>
<tr>
<th>Month</th>
<th>Mean monthly flow (m³/s) Farrell Creek</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>0.03</td>
</tr>
<tr>
<td>Feb</td>
<td>0.02</td>
</tr>
<tr>
<td>Mar</td>
<td>0.07</td>
</tr>
<tr>
<td>Apr</td>
<td>3.88</td>
</tr>
<tr>
<td>May</td>
<td>7.35</td>
</tr>
<tr>
<td>Jun</td>
<td>4.62</td>
</tr>
<tr>
<td>Jul</td>
<td>3.19</td>
</tr>
<tr>
<td>Aug</td>
<td>1.65</td>
</tr>
<tr>
<td>Sep</td>
<td>1.24</td>
</tr>
<tr>
<td>Oct</td>
<td>0.47</td>
</tr>
<tr>
<td>Nov</td>
<td>0.16</td>
</tr>
<tr>
<td>Dec</td>
<td>0.05</td>
</tr>
<tr>
<td>Mean Annual</td>
<td>1.90</td>
</tr>
</tbody>
</table>

### Table 3-7  Annual maximum daily peak instantaneous flow estimates

<table>
<thead>
<tr>
<th>Return Period</th>
<th>Annual maximum daily flow (m³/s)</th>
<th>Annual peak inst. Flow (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>354.2</td>
<td>472.8</td>
</tr>
<tr>
<td>100</td>
<td>290.9</td>
<td>379.5</td>
</tr>
<tr>
<td>50</td>
<td>234.7</td>
<td>299.3</td>
</tr>
<tr>
<td>20</td>
<td>170.4</td>
<td>210.7</td>
</tr>
<tr>
<td>10</td>
<td>128.3</td>
<td>155.0</td>
</tr>
<tr>
<td>5</td>
<td>91.3</td>
<td>107.6</td>
</tr>
<tr>
<td>2</td>
<td>47.8</td>
<td>54.6</td>
</tr>
<tr>
<td>Mean Annual</td>
<td>63.3</td>
<td>77.0</td>
</tr>
</tbody>
</table>

### 3.2.2 Increases to Flood Flow as a Result of Climate Change

Based on a recent climate change impact assessment for the peace Region (NHC, 2017), it is plausible that the 200-year peak inflow to Farrell Creek could increase by 20% by the end of the 21st Century. Once the Site C Reservoir is created, water flowing within the embayment beneath the bridge will be deep with very low velocity, even during large floods. Potential increases in flood flow due to climate change will result in no change in depth and only negligible change in velocity, and thus will have no bearing on the design of the Farrell Creek bridge.
4.0 RESERVOIR ICE REGIME

4.1 Ice Growth and Deterioration

Experience on northern lakes suggests that an ice cover will form after the reservoir water cools to 4 °C and when winds and air temperatures are sufficiently low. While the ice cover is forming, wind could destabilize it and cause individual ice sheets to override each other or drive them against the shore.

During the freeze up period, water levels in the reservoir will likely fluctuate within the normal operating range of El. 460.0 m to El. 461.8 m. The fluctuating water levels would cause a thickening of the ice along the shore line due to the cyclic flooding of shorefast ice located between the bank and the hinge cracks that form between the shore line and the floating ice sheet. However, the ice growth within the large floating ice sheet likely would not be much different than that in a stationary ice sheet.

Ultimately, the Farrell Creek embayment will become totally ice covered and a stable ice cover will develop in most years. The portion of the reservoir within the current Peace River channel may not become fully ice covered.

Snow will accumulate on the ice cover and it will either enhance ice growth by melting and refreezing on the ice surface or, it will reduce the rate of growth due to its insulating effects. The depth of the snow over the ice will change in response to air temperature and solar radiation, and due to compaction. Given the expected low wind speeds it is unlikely that the snow will be redistributed by wind to a significant degree.

In the late winter and early spring, ice deterioration due to absorption of solar radiation will not occur until the snow melts. Once the snow has melted, ice sheets will begin to thin and loose strength. At some point, winds will be strong enough to mobilize the ice sheet, break it into smaller ice floes, and drive those floes against the shoreline and highway embankments.

4.2 Ice Thickness

The design ice thickness for the Farrell Creek embayment is determined by frequency analysis of annual ice thickness over a given period of record. Where measured ice thickness data is not available, as is the case here, ice growth models are necessary to estimate annual ice thickness. These models are based on several simplifying assumptions and require definition of several parameters by calibration against measured ice thicknesses.

Ice growth modelling is comprised of three important and distinct components: (i) the simulation of pre-freeze up water temperatures and the determination of the onset of freeze up; (ii) the under-ice water temperature where it may be relevant; and, (iii) the simulation of the growth of the ice cover over winter and its deterioration in the spring. Ice growth modelling was carried out during the Definition Design (NHC, 2011) and the ice thickness quantiles derived from that analysis are shown in Table 4-1.
### Table 4-1  Ice thickness quantiles for Farrell Creek embayment

<table>
<thead>
<tr>
<th>Quantile</th>
<th>Maximum Ice thickness (Mid-Winter) (m)</th>
<th>Spring Ice thickness (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 year</td>
<td>0.46</td>
<td>0.40</td>
</tr>
<tr>
<td>10 year</td>
<td>0.63</td>
<td>0.57</td>
</tr>
<tr>
<td>20 year</td>
<td>0.68</td>
<td>0.62</td>
</tr>
<tr>
<td>50 year</td>
<td>0.73</td>
<td>0.67</td>
</tr>
<tr>
<td>100 year</td>
<td>0.77</td>
<td>0.71</td>
</tr>
</tbody>
</table>

### 4.3 Ice Strength

#### 4.3.1 Mid-Winter Events

The mid-winter ice events that could produce large thermal or uplift forces on structures would occur under cold conditions when ice movement occurs at temperatures well below its melting point. For these situations, S6-14 suggests that a crushing ice strength of 1,500 kPa be adopted. Crushing strength has a significant temperature-related component that will not translate into a correspondingly high flexural or bending strength. The literature indicates that for cold mid-winter conditions, the upper limit of flexural strength is less dependent on temperature and is affected more by crystal size. So, for lake ice with relatively large crystals, the recommended flexural strength is 1,000 kPa.

#### 4.3.2 Spring Events

Spring ice strength is significantly lower than mid-winter ice strength, but it is more difficult to define explicitly. The ice cover begins to deteriorate when the snow cover melts, the surface of the ice cover becomes exposed to the atmosphere, and there is a net influx of energy (sensible heat and solar radiation) from the atmosphere into the ice cover. At this point, the ice begins to thin and loose strength. The date at which the ice cover becomes unstable depends on its initial thickness, the extent of its deterioration, and the occurrence of winds that are enough to break up and mobilize the ice cover. A crushing ice strength of 700 kPa is recommended for spring events. Measurements indicate that, when ice is at the melting point, the flexural strength of ice is about 84% of the crushing strength (Ashton, 1986), so the corresponding flexural strength would be 590 kPa.

For the situation where large spring floes interact with a shoreline over much larger scales – say the diameter of the floe – it is appropriate to adopt a lower crushing strength of 400 kPa. In this case, the corresponding flexural strength would be 340 kPa.
5.0 HYDRAULIC ANALYSIS

Prior to reservoir filling the proposed bridge will be subject to flows within Farrell Creek that reflect current conditions. A two-dimensional numerical model of Farrell Creek (including a realigned channel and granular berms) has been developed to simulate hydraulics during the construction period.

Telemac-2-D (Hervouet, 2007) was used to model the creek at the bridge, which provides depth and velocity at the mesh nodes. Data was obtained from a previous MIKE11 model of the water surface elevations of Peace River, terrain topography from a 2012 BC Hydro LiDAR survey and design surfaces provided by Binnie and WSP.

5.1.1 Hydraulic Model

The model reach extends from approximately 1.2 km upstream of the proposed bridge to 0.6 km downstream from the confluence with the Peace River. The upstream boundary is located where the flow is contained within the channel, rather than spreading across a wide floodplain.

The model mesh contains over 299,851 nodes and 598,589 elements. Node spacing varies from 0.50 m within the riprap protection area, 1.00 m in the vicinity of the potentially flooded area and to 5.00 m elsewhere. Terrain topographic survey was used for the overbanks of the channel from the existing Highway 29 bridge to the confluence with the Peace River. LiDAR was use for the channel and the Peace River reach of the model.

Surface roughness coefficients (Manning’s n values) represents the flow resistance due to various sources of energy loss. General roughness values used in the model are summarized in Table 5-1.

Table 5-1  Bed roughness resistance values

<table>
<thead>
<tr>
<th>Description</th>
<th>Manning’s n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Riverbed - Peace River</td>
<td>0.024</td>
</tr>
<tr>
<td>Riverbed – Farrell Creek</td>
<td>0.045</td>
</tr>
<tr>
<td>Riverbed – Suggested Realigned Channel</td>
<td>0.035</td>
</tr>
<tr>
<td>Bank riprap - Suggested Realigned Channel</td>
<td>0.05</td>
</tr>
<tr>
<td>Forest</td>
<td>0.110</td>
</tr>
<tr>
<td>Brush</td>
<td>0.050</td>
</tr>
<tr>
<td>Grass</td>
<td>0.030</td>
</tr>
<tr>
<td>Unvegetated overbank areas</td>
<td>0.025</td>
</tr>
</tbody>
</table>
### 5.1.2 Model Scenarios

Two flow scenarios were modelled:

1. 10-yr peak instantaneous flow (Table 3-7);
2. Peak instantaneous mean annual flood (MAF) (Table 3-7).

The model upstream boundary condition is the Farrell Creek flow, and the downstream boundary condition is the Peace River WSE. Peace River WSE was determined using a rating curve from BC Hydro’s Peace River MIKE 11 model. Table 5-2 shows a summary of the model flow scenarios and boundary conditions.

**Table 5-2  Summary of Farrell Creek model boundary conditions**

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Upstream model boundary Farrell Creek inflow</th>
<th>Downstream model boundary WSE at Peace River confluence</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Return Period Flow</td>
<td>Flow (m³/s)</td>
</tr>
<tr>
<td>1</td>
<td>10-yr peak instantaneous</td>
<td>155.0</td>
</tr>
<tr>
<td>2</td>
<td>MAF</td>
<td>77.0</td>
</tr>
</tbody>
</table>

Notes:

a WSE’s are different for each scenario as Farrell Creek flow is added to Peace River flow.

### 5.1.3 Model Results

Results from the model for each scenario through the bridge opening are presented in Table 5-3.

**Table 5-3  Summary of model results**

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Description of Farrell Creek Flow</th>
<th>WSEa (m)</th>
<th>Depth Averaged Velocity (m/s)b</th>
<th>Maximum flow depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bridge opening</td>
<td>Bridge opening</td>
</tr>
<tr>
<td>1</td>
<td>10-yr peak instantaneous</td>
<td>443.2</td>
<td>3.4</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>Mean Annual Flood</td>
<td>442.8</td>
<td>2.2</td>
<td>1.6</td>
</tr>
</tbody>
</table>

Notes:

aWSE through centerline of bridge of bridge opening

bBridge opening average velocities were the peak average velocities along the realigned channel.
Figures presenting the model results are provided below. **Figure 5-1** and **Figure 5-2** present the water surface elevation; **Figure 5-3** and **Figure 5-4** present the depth-averaged velocity; and **Figure 5-5** and **Figure 5-6** present the flow depth for Scenario 1 and Scenario 2 respectively.
Job: 3002853  Date: 11-DEC-2019

Detailed design Farrell Creek 2D modelling
10 year return period
Q = 155.0 m³/s
Water Surface El.

DATA SOURCES: Results from depth averaged 2D model (Telemac-2D) developed based on LiDAR and proposed riprap protection.
Modelled conditions: Farrell Cr at 10 yr flow of 155.0 m³/s; Peace R. at 2 yr flow of 2069 m³/s; and Downstream water level = 442.45 m

Coordinate System: NAD 1983 UTM ZONE 10N
Units: METERS

SCALE - 1:1,200

Protection design
Bridge piers and abutments

Water Surface Elevation (m)

450.00 - 451.00
449.00 - 450.00
448.00 - 449.00
447.00 - 448.00
446.80 - 447.00
446.60 - 446.80
446.40 - 446.60
446.20 - 446.40
446.00 - 446.20
445.80 - 446.00
445.60 - 445.80
445.40 - 445.60
445.20 - 445.40
445.00 - 445.20
444.80 - 445.00
444.60 - 444.80
444.40 - 444.60
444.20 - 444.40
444.00 - 444.20
443.80 - 444.00
443.60 - 443.80
443.40 - 443.60
443.20 - 443.40
443.00 - 443.20
442.80 - 443.00
442.60 - 442.80
442.40 - 442.60
442.20 - 442.40
442.00 - 442.20

Flow

Crest of riprap protection

Crest of riprap protection
**Farrel Creek 2D modelling**

**Mean Annual Discharge - MAF**

\[ Q = 77.0 \text{ m}^3/\text{s} \]

**Water Surface EL.**

**DATA SOURCES:** Results from depth averaged 2D model (Telemac-2D) developed based on LiDAR and proposed riprap protection.

**Modelled conditions:** Farrel Ck at MAF flow of 77.0 m³/s; Peace R. at 2-yr flow of 2069 m³/s; and Downstream water level = 442.39 m
DATA SOURCES: Results from depth averaged 2D model (Telemac-2D) developed based on LiDAR and proposed riprap protection. Modelled conditions: Farrell Creek at 10-yr flow of 155.0 m³/s; Peace R. at 2-yr flow of 2069 m³/s; and Downstream water level = 442.45 m

SCALE - 1:1,200

Coordinate System: NAD 1983 UTM ZONE 10N
Units: METERS

Job: 3002853 Date: 11-DEC-2019

Detailed design
Farrell Creek 2D modelling
10 year return period
Q = 155.0 m³/s Flow
Depth Averaged Velocity

FIGURE 5-3
Study Area

Flow Velocity (m/s)

<table>
<thead>
<tr>
<th>Flow Velocity (m/s)</th>
<th>Height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 0.2</td>
<td>2.8 - 3.0</td>
</tr>
<tr>
<td>0.2 - 0.4</td>
<td>3.0 - 3.2</td>
</tr>
<tr>
<td>0.4 - 0.6</td>
<td>3.2 - 3.4</td>
</tr>
<tr>
<td>0.6 - 0.8</td>
<td>3.4 - 3.6</td>
</tr>
<tr>
<td>0.8 - 1.0</td>
<td>3.6 - 3.8</td>
</tr>
<tr>
<td>1.0 - 1.2</td>
<td>3.8 - 4.0</td>
</tr>
<tr>
<td>1.2 - 1.4</td>
<td>4.0 - 4.2</td>
</tr>
<tr>
<td>1.4 - 1.6</td>
<td>4.2 - 4.4</td>
</tr>
<tr>
<td>1.6 - 1.8</td>
<td>4.4 - 4.6</td>
</tr>
<tr>
<td>1.8 - 2.0</td>
<td>4.6 - 4.8</td>
</tr>
<tr>
<td>2.0 - 2.2</td>
<td>4.8 - 5.0</td>
</tr>
<tr>
<td>2.2 - 2.4</td>
<td>5.0 - 5.2</td>
</tr>
<tr>
<td>2.4 - 2.6</td>
<td>5.2 - 5.4</td>
</tr>
<tr>
<td>2.6 - 2.8</td>
<td>&gt; 5.4</td>
</tr>
</tbody>
</table>

Data Sources: Results from depth averaged 2D model (Telemac-2D) developed based on LiDAR and proposed riprap protection. Modelled conditions: Farrell Ck at MA flow of 77.0 m³/s; Peace R. at 2-yr flow of 2069 m³/s; and Downstream water level = 442.39 m.

Detail design

Farrell Creek 2D modelling

Flow Velocity

Mean Annual Discharge - MAF

Q = 77.0 m³/s

Depth Averaged Velocity

FIGURE 5-4

Scale: 1:1,200

Coordinate System: NAD 1983 UTM ZONE 10N

Units: METERS

Job: 3002853 Date: 11-DEC-2019
DATA SOURCES: Results from depth averaged 2D model (Telemac-2D) developed based on LiDAR and proposed riprap protection. Modelled conditions: Farrell Ck at 10 yr flow of 155.0 m³/s; Peace R. at 2-yr flow of 2069 m³/s; and Downstream water level = 442.45 m. Coordinate System: NAD 1983 UTM ZONE 10N Units: METERS SCALE - 1:1,200 ± 0 25 50 Metres

Flow Depth (m)

<table>
<thead>
<tr>
<th>Flow Depth</th>
<th>Protection design</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 0.2</td>
<td>2.6 - 2.8</td>
</tr>
<tr>
<td>0.2 - 0.4</td>
<td>2.8 - 3.0</td>
</tr>
<tr>
<td>0.4 - 0.6</td>
<td>3.0 - 3.2</td>
</tr>
<tr>
<td>0.6 - 0.8</td>
<td>3.2 - 3.4</td>
</tr>
<tr>
<td>0.8 - 1.0</td>
<td>3.4 - 3.6</td>
</tr>
<tr>
<td>1.0 - 1.2</td>
<td>3.6 - 3.8</td>
</tr>
<tr>
<td>1.2 - 1.4</td>
<td>3.8 - 4.0</td>
</tr>
<tr>
<td>1.4 - 1.6</td>
<td>4.0 - 4.2</td>
</tr>
<tr>
<td>1.6 - 1.8</td>
<td>4.2 - 4.4</td>
</tr>
<tr>
<td>1.8 - 2.0</td>
<td>4.4 - 4.6</td>
</tr>
<tr>
<td>2.0 - 2.2</td>
<td>4.6 - 4.8</td>
</tr>
<tr>
<td>2.2 - 2.4</td>
<td>4.8 - 5.0</td>
</tr>
<tr>
<td>2.4 - 2.6</td>
<td>&gt; 5.0</td>
</tr>
</tbody>
</table>

Study Area

Flow

Crest of riprap protection

Crest of riprap protection
FIGURE 5-6

Date: 11-DEC-2019

Coordinate System: NAD 1983 UTM ZONE 10N
Units: METERS
SCALE - 1:1,200

DATA SOURCES: Results from depth-averaged 2D model (Telemac-2D) developed based on LiDAR and proposed riprap protection. Modeled conditions: Farrell Ck at MAF flow of 77.0 m³/s, Peace R. at 2-yr flow of 2069 m³/s, and Downstream water level = 442.39 m

Protection design
Bridge piers and abutments

Flow Depth (m)

- 0.0 - 0.2
- 0.2 - 0.4
- 0.4 - 0.6
- 0.6 - 0.8
- 0.8 - 1.0
- 1.0 - 1.2
- 1.2 - 1.4
- 1.4 - 1.6
- 1.6 - 1.8
- 1.8 - 2.0
- 2.0 - 2.2
- 2.2 - 2.4
- 2.4 - 2.6
- > 2.6

Farrell Creek 2D modelling
Mean Annual Discharge - MAF
Q = 77.0 m³/s
Flow Depth
6.0 RESERVOIR WAVES

6.1 Wind Waves

Wind-generated waves at the Farrell Creek embayment were assessed by means of two dimensional numerical modelling using the SWAN wave model developed at the Delft University of Technology, Netherlands.

6.1.1 SWAN Model Bathymetry

The underlying bathymetry of the wave model consists of LiDAR of the Peace River Valley obtained from BGC (BGC, 2012) and subsequently gridded at a resolution varying from 2 m to 10 m.

Wave simulations were performed considering a water surface elevation of El. 461.8 m, which corresponds to the reservoir’s MNRL.

6.1.2 Grid Resolution

Two levels of grid resolution were considered for this assessment, a coarser model with a 50 m resolution covering the entire reservoir, and a set of finer nested models with a resolution of 10 m covering the vicinity of specific sites like the proposed bridge and its approaches. These nested models, which apply the results of the coarser model as a boundary condition, were considered to ensure that wave propagation and transformation were sufficiently resolved.

6.1.3 Wind Inputs

The model was operated in “stationary” mode. Stationary mode assumes a wind field to blow indefinitely while in nonstationary mode the field wind varies in time. A summary of wind field inputs for average recurrence intervals (ARIs) are provided in Table 3-4. Wind direction was set specific to the site, inline with local fetches, in order to produce most adverse wave conditions.

6.1.4 Preliminary SWAN model results – Farrell Creek

Results from the wave simulations are summarized in Table 6-1 and Table 6-2 reported at four observation points in the vicinity of the project site (Figure 6-1). Largest waves are generated during wind events from the southwest (Figure 6-2), mostly affecting the shorelines directly west of the creek mouth (Observation Pt. 1) and the south side of the bridge (Observation Pt. 2). Slightly milder but more direct waves affect the east bank during a southerly event (Figure 6-3). The west bridge abutment (Observation Pt. 4) is directly exposed to milder waves during easterly events (Figure 6-4). Larger waves reach that location during a southerly event but less directly (i.e., at a high angle of incidence). Waves reaching the north side of the bridge are relatively small (Figure 6-5).
Table 6-1  Farrell Creek wave simulation results at observation points 1, 2, 3, and 4 (Figure 6-1)

<table>
<thead>
<tr>
<th>Simulation ID</th>
<th>General Wind Direction</th>
<th>ARI</th>
<th>Observation Pt. 1</th>
<th>Observation Pt. 2</th>
<th>Observation Pt. 3</th>
<th>Observation Pt. 4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$H_s$ (m)</td>
<td>$T_p$ (sec)</td>
<td>Dir (°)</td>
<td>$H_s$ (m)</td>
</tr>
<tr>
<td>FC-Prod-E-100_002</td>
<td>E</td>
<td>100</td>
<td>0.34</td>
<td>2.34</td>
<td>110</td>
<td>0.28</td>
</tr>
<tr>
<td>FC-Prod-E-200_002</td>
<td>E</td>
<td>200</td>
<td>0.35</td>
<td>2.34</td>
<td>110</td>
<td>0.29</td>
</tr>
<tr>
<td>FC-Prod-ESE-200_001</td>
<td>SSE</td>
<td>200</td>
<td>0.31</td>
<td>2.09</td>
<td>114</td>
<td>0.27</td>
</tr>
<tr>
<td>FC-Prod-S-200_002</td>
<td>S</td>
<td>200</td>
<td>0.63</td>
<td>2.34</td>
<td>195</td>
<td>0.63</td>
</tr>
<tr>
<td>FC-Prod-SW-100_002</td>
<td>SW</td>
<td>100</td>
<td>0.76</td>
<td>2.93</td>
<td>243</td>
<td>0.79</td>
</tr>
<tr>
<td>FC-Prod-SW-200_002</td>
<td>SW</td>
<td>200</td>
<td>0.80</td>
<td>3.29</td>
<td>243</td>
<td>0.83</td>
</tr>
<tr>
<td>FC-Prod-NNE-200_001</td>
<td>NNE</td>
<td>200</td>
<td>0.13</td>
<td>2.10</td>
<td>76</td>
<td>0.24</td>
</tr>
</tbody>
</table>

Notes:
1. $H_s$ and $T_p$ symbols for significant wave height and period, respectively.

Figure 6-1  Location of numerical observation points in vicinity of Farrell Creek project site.
Figure 6-2  200-yr SW event wave modelling results at Farrell Creek

Figure 6-3  200-yr S event wave modelling results at Farrell Creek
Figure 6-4  200-yr E event wave modelling results at Farrell Creek

Figure 6-5  200-yr NNE event wave modelling results at Farrell Creek
6.1.5  Sensitivity Tests

The model has been mainly developed and refined for wave prediction in open bodies of water such as the North Sea. Because of the different geographical setting of the project sites, several sensitivity tests were performed to assess the variability of the model’s predictions using various calculation parameters. Results from this sensitivity analysis are not presented in this document for sake of brevity.

Since no field measurements exist to justify the selection of specific parameters, it is recommended that model settings resulting in the most adverse conditions be considered for design purposes. Simulation results of significant wave height and peak wave period reported in are increased by 30% and 15%, respectively, based on the results of model sensitivity analysis (Table 6-2).

<table>
<thead>
<tr>
<th>Site</th>
<th>Design Component</th>
<th>Wave Design Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Farrell Creek Bridge</td>
<td>East valley wall S of bridge</td>
<td>Hs (m)</td>
</tr>
<tr>
<td></td>
<td>East valley wall N of bridge</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>West valley wall at bridge opening</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.8</td>
</tr>
</tbody>
</table>

6.2  Vessel Waves

6.2.1  Full Reservoir Level

The design vessel for wave erosion is assumed to be a large recreational cabin cruiser weighing approximately 6,000 kg. The wave height generated by this vessel was determined using the method developed by Maynord (2005). The maximum wave height occurs when the boat is semi-planing and decreasing or increasing the boat speed from this condition causes a reduction in the wave height. Test data suggests that a semi-planing speed of 3 m/s (5.8 knots) maximizes the wave height. Wave height also increases as the distance to the shoreline decreases. In Maynord’s study 9 m was the minimum shoreline distance observed and this value was adopted for this study; it is considered unlikely that boat operators would come much closer than that to an armoured shoreline. With the stated parameters and assumptions, the maximum wave from the design vessel was estimated to have a height of 1 m and period of 2.9 s.

6.3  Landslide Generated Waves (LGWs)

Geotechnical studies conducted by BGC (2012) found that a hypothetical landslide with a return period of 10,000 years occurring along the opposite bank from the outlet of Farrell Creek into the Peace River “Case B” 1 km downstream, could produce LGWs that could impact the new bridge substructure.
Table 6-3 shows estimated parameters for LGWs originating from Case B landslide complex (BGC, 2012; 2017; 2018).

Table 6-3  Initial wave characteristics for Farrell Creek landslide Case B

<table>
<thead>
<tr>
<th>Return Period (yrs)</th>
<th>Wave Amplitude (m)</th>
<th>Wave Period (sec)</th>
<th>Landslide Width (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10,000</td>
<td>15.2</td>
<td>15.2</td>
<td>177</td>
</tr>
<tr>
<td>1,000</td>
<td>9.6</td>
<td>11.3</td>
<td>177</td>
</tr>
</tbody>
</table>

During functional design, Flow-3D software was used to analyse what effect a solitary wave, associated with a 1,000- and 10,000-yr return periods, would have on the short bridge option for the Farrell Creek crossing (NHC, 2019). For detailed design the 10,000-yr LGW was modelled with the following elements:

- Farrell Creek – new bridge piers, associated granular berms, realigned Farrell Creek channel and construction access berms

The model of the proposed bridge estimated the maximum water surface elevation at the bridge of El. 472.9 m for the 10,000-yr wave. The corresponding maximum wave amplitudes at the bridge is 11 m and velocity was 16.0 m/s.

Results are detailed in the memo of the LGW results, provided in Appendix C.

Typical LGW management strategies recommend drawing down a reservoir to reduce the potential impact an LGW would have on bridge superstructure (i.e. reduce wave height and wave forces). The 10,000-yr wave with drawn down reservoir elevation of 446.5 m was assessed at the Halfway River segment. Reducing the water elevation resulted in a larger initial wave amplitude at the reference point in the vicinity of the landslide impact point, and increased velocity when compared to the MNRL scenario. At the west highway approach the lower reservoir level enabled the wave to run-up along the existing ground over a longer distance than during the MNRL scenario resulting in a higher wave elevation overtopping the highway. Additionally, the LGW at 446.5 m reservoir elevation resulted in higher forces at the western piers (NHC, 2019).

No modelling of the LGW with drawn down reservoir level has been carried out at Farrell Creek. It is anticipated that an increase in initial wave height and velocity will also occur for the Case B LGW by lowering the reservoir level. The western abutment and piers 1 and 2 at Farrell Creek have similar exposure to the LGW as the western abutment and piers at Halfway, so there may be a similar increase in pier forces and run-up.
7.0 HYDRAULIC DESIGN

7.1 Pier 1 Berm and Pier 2 Isolation Berm During Construction

Pier 1 berm and Pier 2 may be exposed to flowing water during construction and require protection. At the Pier 1 berm riprap will tie into high ground upstream of the transition from the existing west valley wall to Pier 1 berm, around the berm and extending downstream a short distance along the existing west valley wall. The Pier 2 isolation berm will consist of a riprap lined berm (with 2H:1V side slopes, crest width of 2 m and elevation minimum 0.3 m above the 10-year flow) that surrounds the Pier 2 construction area and ties into the east floodplain. The design is detailed on drawings 2184-281 and 2184-283 (Appendix B). The construction period is expected to last 1 to 2 years and the recommended design return period for construction conditions is 10 years.

7.2 Farrell Creek Realignment During Construction

In order to ensure construction of Pier 1 and Pier 2 occur in the dry, Farrell Creek will have to be realigned and confined to a single channel slightly east of its present location. Responsibility for the realigned channel will fall to the contractor, however, a suggested realigned channel is detailed on the T3 Reference drawing (Appendix B). The construction period is expected to last 1 to 2 years and the design criteria for the realigned channel are as follows:

- a. Design (10-year Annual Peak) Flow: 155 m³/s
- b. Minimum Channel Bottom Width: 20 m
- c. Minimum Freeboard (to Top of Berm): 0.3 m
- d. Banks: Riprap armoured at 2H:1V
- e. Low Flow Channel (Notch): 5 m Width x 0.3 m Depth

A temporary bridge may have to be installed across the realigned channel to provide access to a gravel source during construction. The temporary riprap protection around Pier 1 berm and the Pier 2 isolation berm are mandatory design requirements and not within the contractors realignment design scope.

7.3 Scour During the Construction Period

7.3.1 Contraction Scour

Contraction scour occurs when water flows from a relatively wide channel and floodplain (floodway) in the approach channel into a relative narrow bridge waterway opening that is constricted by bridge end fills, abutments, piers, etc.
Flow is constricted at the proposed bridge due to the Pier 2 temporary works, however the floodplain upstream of the pier is narrow and conveys a negligible amount of flow during the construction period – the potential for contraction scour is low.

Contraction scour may occur to some degree at the temporary bridge crossing. The Contractor’s hydrotechnical engineer will be responsible for assessing and mitigating the scour risk in this case.

### 7.3.2 Natural Scour

Natural scour occurs in streams even if the channel is not constricted or controlled to a significant degree. The causes of natural scour can include: (i) an unusually large flood; (ii) accelerated, deep flow along the outside of a bend; (iii) lateral shifting of the channel thalweg; (iv) flow alongside or impinging upon rock outcrops, debris jams, other hard points or rigid materials along the channel boundaries; and iv) sudden concentration of flow such as the confluence of two or more channels.

There will be natural scour potential along the toe of the Pier 1 berm and Pier 2 isolation berm riprap. The 10-yr natural scour elevation in the realigned channel has been estimated using the Blench’s Regime Depth Method (Blench, 1957; Transportation Association of Canada, 2004) with the following inputs:

- Design flow = 155.0 m³/s (10-yr return period flow)
- Water surface width = 30 m
- Realigned channel slope = 0.005
- Median diameter of bed material = 10 mm
- Z factor (scour depth multiplier applied to regime depth) = 2.0
- 10-yr water surface elevation = El. 443.2 m
- Streambed elevation = El. 441.2 m

Based on the above inputs, the 10-yr scour depth is 3.0 m below the 10-yr flood elevation. Pier 1 berm and Pier 2 isolation berm riprap will be keyed in below the natural scour elevation so that it remains stable under design scour conditions.

There will be natural scour potential along the toe of riprap placed in the realigned channel. The Contractor will be responsible for estimating and designing for such scour.

### 7.3.3 Local Scour

Local scour occurs as a result of flow striking and then accelerating around both sides of an obstruction within the floodway such as bridge piers. Since streamflow velocities will be negligible during normal reservoir operations and flow during the construction period will be confined within the realigned channel the potential for local scour is low and not applicable for pier design.
7.4 Temporary Bridge for Gravel Extraction

A temporary, clear span bridge may be required to cross the channel upstream of the existing Highway 29 bridge to access a gravel source. The temporary bridge soffit must be 0.5 m above the predicted 10-yr flood elevation with a 20 m bottom width and 2H:1V side slopes.

7.5 10,000-yr LGW Forces

The 10,000-yr Case B LGW reaches the Farrell Creek bridge at approximately 68 s. Piers 4 was impacted by the wave first (at about 70 s), followed by Pier 5 at 71, Pier 3 at 72 s, Pier 2 at 77 s and Pier 1 at 79 s. The LGW does not impact the bridge abutments or the girders. An elevation view of the maximum water surface elevation along the highway embankment is shown in Figure 7-1. As it passes under the bridge, the LGW reaches a maximum water surface elevation of approximately 472.9 in the vicinity of the west abutment, corresponding to a maximum wave height of just over 11.0 m.

Hydrodynamic forces at the piers caused by pressure forces from the 10,000-yr LGW were assessed at each pier of the new bridge. Pressure forces at the piers are due to the pressure differential between opposite sides of the pier caused by the passage of the LGW. Detailed FLOW3D analysis and results of the peak horizontal force exerted by the 10,000-yr LGW at the bridge piers is provided in Appendix C.

A summary of the horizontal forces acting on the bridge piers is shown in Table 7-1. Figure 27 in Appendix C provides a visual representation of the horizontal forces with respect to the bridge pier.

After the LGW impacts Pier 5 it is reflected towards the existing bank and then subsequently reflected back into the reservoir. This causes a secondary, weaker wave to impact the bridge piers. The force caused by the reflected wave is smaller than the peak force (ie. 30 to 40% of the peak), however, the force direction also switches from northwest to southwest. In particular, when the peak force impacts Pier 2 in a northwest direction, the secondary force occurs at Pier 4 in the southwest direction.
### Figure 7-1   Maximum water surface elevation at the new bridge

### Table 7-1   Summary of peak horizontal forces exerted on the bridge piers by the 10,000-yr Case B LGW

<table>
<thead>
<tr>
<th>Pier</th>
<th>$T^1$ (s)</th>
<th>$F_x^2$ (kN)</th>
<th>$F_y^2$ (kN)</th>
<th>$F_h^2$ (kN)</th>
<th>$\alpha^2$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier 1</td>
<td>79.4</td>
<td>-651</td>
<td>112</td>
<td>661</td>
<td>-80</td>
</tr>
<tr>
<td>Pier 2</td>
<td>75.2</td>
<td>-1,819</td>
<td>272</td>
<td>1,839</td>
<td>-82</td>
</tr>
<tr>
<td>Pier 3</td>
<td>71.7</td>
<td>-1,314</td>
<td>312</td>
<td>1,351</td>
<td>-77</td>
</tr>
<tr>
<td>Pier 4</td>
<td>70.3</td>
<td>-1,409</td>
<td>128</td>
<td>1,415</td>
<td>-85</td>
</tr>
<tr>
<td>Pier 5</td>
<td>71.2</td>
<td>90</td>
<td>1,158</td>
<td>1,162</td>
<td>4</td>
</tr>
</tbody>
</table>

**NOTES:**
1. Time refers to the time of arrival of the wave measured since the occurrence of the landslide.
2. Refer to Appendix C for a definition sketch of force components and direction.
3. Force magnitudes are outputs from the numerical model without any factoring.

### 7.6   Ice Forces on the Permanent Bridge

A 50-yr terminal floe velocity of 0.85 m/s has been adopted for pier design based on a 50-yr Fort St. John wind speed of 24.3 m/s and a fetch length of at least 1400 m. A 10-yr terminal floe velocity of 0.77 m/s has been adopted for drawdown conditions. Wind-induced ice floe velocities are limited by the water drag associated with the floe velocity so longer fetch lengths would not result in higher floe velocities.
once the terminal velocity is achieved. A nominal ice floe diameter of 146 m has been selected, which is about the largest floe that could pass beneath the bridge without contacting more than one bridge pier.

7.6.1 The Concept of Return Period for Ice Forces

S6-14 does not specify a return period to be used for the determination of ice forces. The design ice force is a combination of several variable parameters (ice thickness, ice strength, floe velocity, floe size). There are multiple combinations of these parameters that could produce the same ice force so a simple multiplication of return periods for each cannot be used to determine the return period of the ice force. To simplify the process of selecting an appropriate design ice force, design values are adopted for each of these parameters and the design ice force is calculated from the combination of these values. Selecting say a 100- or 200-yr value for each parameter would result in a too-severe design ice force.

The one parameter that can be most easily assigned a return period is ice thickness. For this study NHC recommends the use of a 50-yr ice thickness (spring or mid-winter value depending on the force scenario), but since there is a complex combination of other variables involved, this should not be seen as a lesser standard than the 100- or 200-yr return period, which are the basis of other bridge design elements.

7.6.2 Force Elevations on the Bridge Piers

Table 7-2 summarizes the pier characteristics at the proposed Western Segments crossings. The normal operating range of the proposed future reservoir is El. 460.0 m to El. 461.80 m and the corresponding minimum and maximum normal ice levels, the levels at which the ice would act on the pier (at center of ice thickness), are 459.7 m and 461.5 m, respectively, which is within the conical section of the pier.

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Farrell Creek</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier shape</td>
<td>Round</td>
</tr>
<tr>
<td>Diameter of bottom part of column (m)</td>
<td>3.5</td>
</tr>
<tr>
<td>Diameter of upper part of column (m)</td>
<td>2.4</td>
</tr>
<tr>
<td>Height of conical transition (m)</td>
<td>2.77</td>
</tr>
<tr>
<td>Cone angle (degrees from horizontal)</td>
<td>78.8</td>
</tr>
<tr>
<td>Bottom elevation of cone (El., m)</td>
<td>459.51</td>
</tr>
<tr>
<td>Top elevation of cone (El., m)</td>
<td>462.29</td>
</tr>
<tr>
<td>Coefficient of friction of cone face</td>
<td>0.15</td>
</tr>
</tbody>
</table>
7.6.3 Dynamic Longitudinal Forces Acting on Bridge Piers

There are two potential mechanisms by which the ice could develop a longitudinal, dynamic force on the pier: (i) flexural failure of the ice sheet as it interacts with the cone and the subsequent pushing of the ice up the cone, and (ii) crushing of the ice sheet against the vertical face of the 2.4 m diameter upper part of the pier or the larger 3.5 m diameter lower part of the pier. The bending failure against the cone is the most likely scenario, but the force and its corresponding moment will vary with the ice level. The largest ice force would occur at the minimum normal ice level where the cone diameter is the largest at 3.42 m. The smallest ice force would occur at the maximum normal ice level where the cone diameter is the smallest at 2.70 m.

While there are several ways to calculate the bending force, including using the approach suggested in S6-14, the most defensible analytical method in the literature is the three-dimensional approach formulated by Ralston (1980). The Ralston technique is not necessarily conservative, but it produces higher forces than the bending force formula given in the S6-14 code and other methods that rely on a two-dimensional approximation of the ice failure characteristics, and it is the approach recommended herein.

Table 7-3 summarize the ice forces on the conical portions of the pier for a range of water and ice levels for the Western Segments. The ice pushing force is less than 10% of the ice bending failure force because the vertical length of the cone is quite short.

<table>
<thead>
<tr>
<th>Water Level (m)</th>
<th>Ice Force Level (El., m)</th>
<th>Cone Diameter (m)</th>
<th>Ice Breaking Force (MN)</th>
<th>Ice Pushing Force (MN)</th>
<th>Total Force (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>460.0</td>
<td>459.72</td>
<td>3.42</td>
<td>9.13</td>
<td>0.93</td>
<td>2.26</td>
</tr>
<tr>
<td>460.6</td>
<td>460.32</td>
<td>3.18</td>
<td>9.07</td>
<td>0.68</td>
<td>2.14</td>
</tr>
<tr>
<td>461.2</td>
<td>460.92</td>
<td>2.94</td>
<td>9.01</td>
<td>0.45</td>
<td>2.02</td>
</tr>
<tr>
<td>461.8</td>
<td>461.52</td>
<td>2.78</td>
<td>8.96</td>
<td>0.24</td>
<td>1.90</td>
</tr>
</tbody>
</table>

Notes:
- a. Force required to cause bending failure in the approaching ice floe.
- b. Force required to push the broken ice up the cone before it sheds laterally off the cone.
- c. Ice force is applied at the mid-point in the ice thickness which is 42% of the ice thickness below the water for the specific gravity of ice of 0.92.
- d. Lower of bending force and crushing force.

Given that the maximum ice level is very close to the top of the cone, the ice could ride up and fail in crushing against the 2.4 m diameter vertical pier. This force would act at El. 462.6 m. The full crushing force associated with a floe diameter of 146 m and a floe velocity of 0.85 m/s would be 1.74 MN.
The bottom of the cone is about 0.2 m above the lower limit of the normal ice action of 459.3 m so a crushing force of 0.56 MN could develop on the 3.5 m diameter pier at elevation 459.4 m. This force is not energy limited because the total ice thickness contributing to the kinetic energy of the floe is greater than the contact thickness of 0.2 m.

During a drawdown event the ice level may drop below the level of the cone so dynamic ice forces may occur on the 3.5 m diameter portion of the pier. A 10-yr spring ice thickness of 0.57 m and a 10-yr terminal floe velocity of 0.77 m/s is recommended for this type of event. The corresponding energy limited crushing force for the 3.5 m diameter pier is 1.79 MN.

The direction of the ice forces will depend upon the wind direction. From the configuration of the embayment, it is likely that the ice floes will approach the piers from an angle between -45° to +45° from a line perpendicular to the bridge centreline. Therefore, the ice force would produce simultaneous force components that are perpendicular and parallel to the bridge. While the S6-14 code suggests that an additional force transverse to the direction of the main force be applied concurrently with the main force for round pier noses on rectangular piers, there is no need to do so for round piers. For round piers, as the pier crushes through the ice sheet, the transverse forces are symmetrical around the pier, with the forces offsetting each other.

### 7.6.4 Uplift and Downdrag Forces Acting on Bridge Piers

The S6-14 code prescribes the equations to be used in calculating the design uplift/ downdrag forces. The actual forces are dependent on: (i) the ice thickness; (ii) the flexural strength of the ice sheet; (iii) its modulus of elasticity; and (iv) the magnitude of the water level fluctuations. In applying the S6-14 equations, it is assumed that: (i) water level fluctuations are a rapid enough to allow the ice sheet to fail in flexure; (ii) the time between water level fluctuations is long so that the ice can adhere fully to the pier; (iii) the worst-case fluctuations occur in late winter when the ice is at its thickest; and (iv) the ice sheet is internally sound and its temperature is below its melting point. These assumptions lead directly to the adopted ice characteristics shown in Table 7-4.

#### Table 7-4  Adopted ice characteristics for uplift and downdrag forces

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Farrell Creek</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushing strength of ice sheet (kPa)</td>
<td>1500</td>
<td>Ice sheet is internally sound, and its temperature is below melting</td>
</tr>
<tr>
<td>Flexural (tensile) strength (kPa)</td>
<td>1000</td>
<td>Typical mid-winter strengths measured in lakes during cold periods</td>
</tr>
<tr>
<td>Elastic modulus (kPa)</td>
<td>1.5 x 10^6</td>
<td>From deflection measurements of ice sheets in northern Alberta</td>
</tr>
<tr>
<td>Ice thickness (m)</td>
<td>0.73</td>
<td>50-yr mid-winter ice thickness for Farrell Creek (Table 4-1)</td>
</tr>
<tr>
<td>Angle of truncated wedge (degrees)</td>
<td>90</td>
<td>See the description in the S6-14 code</td>
</tr>
</tbody>
</table>
The maximum uplift and downdrag forces for the pier would occur when the water level is at El. 460.0 m, the corresponding diameter of the cone and design force for each crossing is provided in Table 7-5.

<table>
<thead>
<tr>
<th>Design Factor</th>
<th>Farrell Creek</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of cone at El. 460.0 m water level</td>
<td>3.43 m</td>
</tr>
<tr>
<td>Design force</td>
<td>1.03 MN</td>
</tr>
</tbody>
</table>

During a drawdown event the ice level may drop below the level of the cone so uplift and downdrag forces may occur on the 3.5 m diameter portion of the pier. A 10-yr mid-winter ice thickness is recommended for this type of event. A summary of the design components and corresponding uplift and downdrag force for each crossing is provided in Table 7-6.

<table>
<thead>
<tr>
<th>Design Factor</th>
<th>Farrell Creek</th>
</tr>
</thead>
<tbody>
<tr>
<td>10-yr mid winter ice thickness</td>
<td>0.63 m</td>
</tr>
<tr>
<td>Design force</td>
<td>0.80 MN</td>
</tr>
</tbody>
</table>

### 7.6.5 Thermal Forces Acting on Bridge Piers

Thermal forces acting laterally on the piers are affected by: (i) the confinement of the surrounding ice sheet; (ii) the thermal response of the ice sheet to changes in air temperature; and, (iii) creep effects which tend to reduce the thermal forces when the ice sheet expands slowly. Thermal expansion forces on piers need to be considered only if there is a possibility that the loads are unbalanced. For example, given the potential for the ice to melt out in the centre of the bridge waterways, a situation may arise where one side of a pier is ice free and thermal forces could develop on the pier by ice spanning between two piers or between an embankment and a pier. Another situation that could result in thermal forces would be expansion of the ice in the embayment out into the ice-free reservoir.

The S6-14 code does not explicitly define the thermal forces that may develop, but experience on long structures suggests that a reasonable upper limit of the force per unit length is a function of the ice...
thickness to the power of 1.5. For structures that are relatively slender, it is appropriate to allow for a concentration of the thermal forces by introducing an indentation coefficient (Carter Consultants, 2001).

In the embayment, the largest thermal force will potentially occur in winter when the ice is at its maximum thickness and there is a large increase in air temperature. Therefore, it is appropriate to adopt the design winter ice thickness in the calculation. It can be assumed that ice thickness is relatively uniform along the bridge length.

**Table 7-7** summarizes the thermal ice forces for a range of winter water levels. The worst-case force acting in any direction would be 0.78 MN at Farrell Creek. These forces are less than the maximum flexural forces that the ice sheet can sustain and thus they are not limited by the flexural strength of the ice sheet.

During a drawdown event the ice level may drop below the level of the cone so thermal forces may occur on the lower portion of the pier. A 10-yr mid-winter ice thickness is recommended for this type of event. A summary of the design components and corresponding thermal force for each crossing is provided in **Table 7-8**.

**Table 7-7**  **Farrell Creek - Thermal ice forces on the conical portions of the piers**

<table>
<thead>
<tr>
<th>Water level (El., m)</th>
<th>Ice force level (El., m)</th>
<th>Cone diameter (m)</th>
<th>Basic thermal force (MN)</th>
<th>Indentation coefficient</th>
<th>Thermal force (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>460.0</td>
<td>459.7</td>
<td>3.43</td>
<td>0.54</td>
<td>1.44</td>
<td>0.78</td>
</tr>
<tr>
<td>460.6</td>
<td>460.3</td>
<td>3.19</td>
<td>0.50</td>
<td>1.46</td>
<td>0.74</td>
</tr>
<tr>
<td>461.2</td>
<td>460.9</td>
<td>2.95</td>
<td>0.47</td>
<td>1.50</td>
<td>0.70</td>
</tr>
<tr>
<td>461.8</td>
<td>461.5</td>
<td>2.71</td>
<td>0.43</td>
<td>1.53</td>
<td>0.66</td>
</tr>
<tr>
<td>&lt;460.0</td>
<td>&lt;459.7</td>
<td>3.50</td>
<td>0.44</td>
<td>1.38</td>
<td>0.61</td>
</tr>
</tbody>
</table>

**Table 7-8**  **Summary of thermal design factors and force for drawdown event.**

<table>
<thead>
<tr>
<th>Design Factor</th>
<th>Farrell Creek</th>
</tr>
</thead>
<tbody>
<tr>
<td>10-yr mid winter ice thickness</td>
<td>0.63 m</td>
</tr>
<tr>
<td>Design force</td>
<td>0.61 MN</td>
</tr>
</tbody>
</table>
7.6.6 Ice Forces on Riprap

For the purpose of estimating ice forces on riprap, local (valley) windspeed estimates are used as opposed to Fort St. John windspeeds. This represents a less conservative approach, which is considered justifiable since the ice loads in this case are not acting directly on the bridge structure. Frequency analysis of hindcast wind data for a location near the mouth of Farrell Creek suggests that a 50% reduction in 50-yr windspeed (relative to Fort St John) is appropriate for estimating dynamic ice forces on riprap (NHC, 2018; EBA, 2010).

There are two major types of ice action that can affect the integrity of riprap placed along the slopes adjacent to Pier 1 and Pier 5 (Farrell Creek) and highway embankment slope (Lynx East and Farrell Creek):

- Plucking of individual rocks off the side slope by rising ice levels when the rocks are imbedded in the ice sheet (ice plucking).
- 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:

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

7.6.6.1 Ice Plucking

The plucking process is like that which produces uplift forces on bridge piers. Ice adheres to the individual rocks and when the water level fluctuates, the ice lifts or pulls the mass of rocks to which it has adhered. The force is based on the maximum vertical force that can be applied to the edge of the sheet without having it fail in flexure, if the ice sheet can be treated as a beam on an elastic foundation. Since this process can occur anytime during the winter, it is reasonable to assume a high flexural strength of 1,000 kPa.

Given the adopted values of the modulus of elasticity and the flexural strength of ice, and if it is assumed that the ice plucks individual rocks, one 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 requirement suggests that the minimum diameter of rock to ensure stability should be in the order of 1.0 m if applied
to northern areas. In fact, substantially smaller rock sizes appear to function very well under the relatively severe ice conditions in the north.

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 mass of rock that is embedded in the ice plus the mass of rock on the slope directly above where the ice level intersects the slope under the riprap. Therefore, plucking resistance is related to the thickness of the riprap layer rather than the actual size of the rock pieces.

**Figure 7-2** shows the relationship between ice thickness and the thickness of the riprap required to resist plucking. For a winter ice thickness of 0.73 m, a side slope of 1.5H: 1V, an efficiency factor of 0.75 (the fraction of theoretical rock volume that resists the force), and a rock density of 2,520 kg/m³, the required riprap thickness to resist plucking is 850 mm. While the plucking resistance is not explicitly related to the size of the individual rocks, if it assumed that the thickness of the riprap is twice the median rock size, the median rock size required to withstand plucking by ice would be a minimum of 425 mm.

![Thickness of Riprap Required to Prevent Disturbance by Plucking](image)

**Figure 7-2** Thickness of riprap required to prevent disturbance by plucking
7.6.6.2 Ice Forces from Moving Ice Sheets

The force applied by a moving ice sheet 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 162 m adopted for the pier analysis is also appropriate for the riprap forces given the geometry of the valley. A terminal floe velocity of 0.68 m/s has been adopted based on a 50-yr valley wind speed of 19.3 m/s.

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 with these being the most likely:

- crushing of the ice sheet against the face of an individual rock and displacing that one rock
- sliding of the floe over the top of the riprap without mobilizing the riprap
- crushing of the ice floe against several rocks so that the ice force is distributed along the embankment

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 (on the order of one rock diameter) it is possible 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 displace locally, the riprap would likely remain intact. In fact, laboratory studies have shown 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.

Sodhi et al. (1996) carried out a set of physical model studies to examine ice/riprap interactions. His work provides an assessment of the effects of a worst case, strength limited (not energy limited) condition on the stability of riprap. The model studies showed that if the horizontal ice force applied on an individual rock (characterized by the $D_{100}$ in Sodhi’s experiments) is resolved into its upslope component and that force is compared to the down-slope component of the weight of the rock, the riprap placement is either totally stable or experiences some minor instability when the ice force is greater than about 60 to 200 times the weight of an individual rock. Since the maximum horizontal ice force is related to the thickness and the flexural strength of the ice sheet, it is possible to define a range of rock sizes that would be stable for the adopted embankment slopes and the adopted design ice characteristics.
Table 7-9 summarizes the worst-case ice forces that could develop for a spring ice thickness of 0.67 m and an ice strength of 340 kPa, and the corresponding rock size that would be stable under those conditions on a 1.5H:1V slope, 2H:1V slope and 3H:1V slope. The forces and rock size for 590 kPa are also provided for comparison. A D$_{50}$ of riprap of 200 mm will prevent extensive displacement due to forces from moving ice sheets even for the higher ice strength.

### Table 7-9  Maximum ice forces and corresponding stable rock sizes

<table>
<thead>
<tr>
<th>Ice Thickness (m)</th>
<th>Flexural Strength (kPa)</th>
<th>Maximum Ice Force (MN/m)</th>
<th>Rock Slope (H:V)</th>
<th>Stable Rock Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Vertical</td>
<td>Horizontal</td>
<td>Up Slope</td>
</tr>
<tr>
<td>0.67</td>
<td>340</td>
<td>7.1</td>
<td>7.8</td>
<td>6.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.1</td>
<td>5.4</td>
<td>4.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.5</td>
<td>4.3</td>
<td>3:1</td>
</tr>
<tr>
<td>590</td>
<td>12.3</td>
<td>13.5</td>
<td>11.2</td>
<td>7.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10.5</td>
<td>9.4</td>
<td>7.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7.8</td>
<td>7.4</td>
<td>3:1</td>
</tr>
</tbody>
</table>

a. Angular rock with 2520 kg/m$^3$ density


### 7.6.6.1 Up-Slope Sliding of Ice

Forces due to the sliding of an ice sheet over the riprap would develop independent of rock sizes because 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. Assuming that the energy required to break the ice in bending is negligible (a conservative assumption), the maximum ‘run-up’ height that the ice will achieve depends on: (i) the kinetic energy of the ice floe (a function of its mass and velocity); (ii) the slope angle of the embankment; (iii) the friction coefficient between the ice and the riprap; (iv) the ice thickness; and (v) the width of the ice being pushed up the slope (some fraction of the diameter of the ice floe). A summary of the design inputs and resultant run-up heights for Farrell Creek is provided in Table 7-10.
Table 7-10  Summary of design factors and run-up height.

<table>
<thead>
<tr>
<th>Design Factor</th>
<th>Farrell Creek</th>
</tr>
</thead>
<tbody>
<tr>
<td>50-yr valley wind speed</td>
<td>19.3 m/s</td>
</tr>
<tr>
<td>Floe diameter</td>
<td>146 m</td>
</tr>
<tr>
<td>Embankment slope</td>
<td>2H:1V</td>
</tr>
<tr>
<td>Run-up height (2H:1V slope)</td>
<td>2.9 m</td>
</tr>
</tbody>
</table>

7.6.7 Effects of Shorefast Ice on Riprap

The presence of thick shorefast ice would tend to shield the embankments from a direct ice force, but the net force on the embankment would still be related to the characteristics of the large ice sheet that would be driven against the embankment. Furthermore, the presence of shore ice cannot be relied upon to shield the embankments in all situations. During break up in the spring, the shore ice would either remain attached to the shore and melt in place or melt away in relatively small pieces, so it is not expected to produce significant forces.

7.7 Erosion protection

The focus of this section is on the use of riprap as a means of erosion protection.

Erosion protection is required during the construction period to protect the Pier 1 berm and Pier 2 isolation berm. The Contractor will be responsible for estimating and design of the riprap within the realigned channel.

During normal operation of the reservoir, erosion of the side slope adjacent to Pier 5 is of concern. The side slopes of the Farrell Creek valley are predominantly weathered bedrock within the wave erosion zone. Erosion of the bedrock from wave and ice action is of concern along the east valley slope at Pier 5 as the abutment just encroaches upon the Stability Impact Line (SIL); per the hydrotechnical design criteria erosion protection is recommended for this scenario. The west abutment is set back roughly 10 to 20 m from the SIL and therefore does not require riprap protection³.

During drawdown of the reservoir erosion of the Pier 1 granular berm will occur.

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³ Additional slope stability assessment was conducted by Wood and further confirmed that riprap is not required along the existing west bank (Wood, 2019)
7.7.1 East Slope Adjacent to Pier 5

Riprap sizing for protection against waves uses the Wave Momentum Flux equation developed by Melby (2005) outlined in the Coastal Engineering Manual (USACE 2011). The main inputs to the equation are as follows:

- Design significant wave height = 1.1 m (Table 6-2);
- Design wave period = 3.8 s (Table 6-2);
- Rock density = assumed at 2,520 kg/m$^3$;
- Damage Coefficient = 2.0; and
- Embankment Slope = 1.5H:1V (highway).

The required D$_{50}$ of riprap to protect against this wave is 500 mm for the highway embankment slopes. The nearest corresponding BCMOT Class of riprap is 250-kg Class for the 1.5H:1V slope. A portion of the riprap may be within possibly acid generating (PAG) material – this riprap will need to be a limestone material. If sourced from West Pine Quarry MCW Zone 6e riprap is acceptable. For wave protection, the minimum thickness of the riprap, measured perpendicular to the slope, should be a minimum of two times the D$_{50}$ resulting in a placed thickness of 1.2 m.

For the wind waves, runup and rundown elevations were calculated using the procedures in the EurOtop (2016). The reservoir elevation for the maximum runup calculation (Delft Hydraulics method) was taken as the MNRL, El. 461.8 m, and the reservoir elevation for maximum rundown calculation (Vandermeer method) was taken as the minimum normal operating level, El. 460.0 m.

The main parameters and assumptions used in the runup and rundown analysis are:

- Water Depth at Embankment Toe taken as 1.0 m
- Roughness factor (for runup) 0.55
- Run-down Permeability Coefficient 0.10
- Embankment slope 1.5H:1V (existing slope)

The maximum runup is 2.2 m and the maximum rundown is 1.0 m along the 1.5H:1V slope.

For the 1.5H:1V existing slope the riprap for wave protection is required to extend from El. 464.6 m (summation of the full supply level, runup, and 0.6 m freeboard) down to El. 458.0 m (the minimum normal operating level, less rundown, less 1.0 m freeboard).

The riprap extending from roughly 30 m north of Pier 5 to 30 m south will be placed along the face of the existing slope with a portion of the riprap toe embedded requiring excavation of the existing highway fill. The riprap extending south of Pier 5 transitions from primarily being embedded into
highway road fill to being notched into the existing bedrock slope. Along the toe of the notched portion the riprap will be angled back into the bedrock at 3H:1V to ensure it doesn’t slide off the slope, and the riprap within the toe depression will be grouted. The condition and material type of the existing bank has not been confirmed with field studies – granular filter between the riprap and existing bank is required.

For layout and extents of riprap placement see drawings 2184-281 to 283 (Appendix B).

7.7.1.1 Granular Filter

A granular filter is required between the riprap and underlying, fine erodible soils, wherever the riprap is exposed to regular wave action and fluctuating reservoir levels. The gradation of granular filters must satisfy the following criterion (Brown and Clyde 1989):

\[
D_{15c}/D_{85f} < 5 < D_{15c}/D_{15f} < 40
\]

Equation 7-1

where \(D_{15}\) and \(D_{85}\) refer to the 15% and 85% sieve passing sizes, and subscripts “c” and “f” refer to the coarse and finer layers, respectively. The criterion is imposed at the interfaces between the underlying material and the filter, and between the filter and the overlying riprap.

For BCMOT Class 250-kg riprap and an underlying native soil or fill, a single granular filter layer with a gradation presented in Table 7-11 and a thickness of 200 mm is required.

Table 7-11 Granular Filter Gradation

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>Percent Passing (%) Western Segments</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>80 - 100</td>
</tr>
<tr>
<td>75</td>
<td>50 – 70</td>
</tr>
<tr>
<td>50</td>
<td>40 – 60</td>
</tr>
<tr>
<td>12.5</td>
<td>5 – 10</td>
</tr>
<tr>
<td>0.6</td>
<td>0 – 5</td>
</tr>
</tbody>
</table>

7.7.2 Drawdown Protection for Pier 1 Granular Berm

Riprap is designed to resist ice forces as detailed in Section 7.5 and small wave action. The recommended riprap is BCMOT Class 50-kg. The slope of the riprap must match the granular berm slope of 2.25H:1V. The thickness should be 0.55 m. The top elevation of the riprap is 447.0 m extending down to existing ground or tying into the realigned channel right berm. No granular filter is required for this riprap. For layout and extents of riprap placement see drawings 2184-281 to 283 (Appendix B).
7.7.3 Riprap for the Pier 1 Berm and Pier 2 Isolation Berm

The sizing of riprap for protection against stream flow follows U.S. Army Corps of Engineers procedures (USACE, 1991; TAC, 2004), which require the following inputs:

- Design velocity = 3.6 m/s;
- Design depth of water along protected slopes = 2.0 m;
- Coefficient of Stability, $C_s = 0.3$ (for angular riprap);
- Velocity distribution coefficient, $C_v = 1.15$;
- Thickness coefficient, $C_t = 1$;
- Side slope factor, $K = 0.9$ (for 2H: 1V slopes); and
- Rock density = 2,520 kg/m$^3$ \(^4\)

The velocity and depth inputs are for the estimated 10-yr flow on Farrell Creek (with no Peace River backwater) and were calculated using outputs from the two dimensional hydraulic model discussed in Section 5.0.

The estimated $D_{50}$ of riprap is 487 mm. The nearest corresponding BCMOT Class is 250-kg. The layout for the riprap is shown on drawings 2184-281 to 283 (Appendix B).

\(^4\) Density for rock from Portage Mountain Quarry
8.0 REFERENCES


BCMOT Supplement to CAN/CSA-S6-14, Bridge Standards and Procedures Manual, Volume 1 Supplement to CHBDC S6-00, June 2006.


Canadian Highway Bridge Design Code CAN/CSA-S6-14 (2006); plus S6S1-10 (Supplement 1; 2010) and S6S2-11 (Supplement 2; 2011)


US Army Corp of Engineers (USACE), 2016. HEC-RAS River Analysis System, Version 5.0.3. Software developed by the Hydrologic Engineering Center, Davis CA.

APPENDIX A

Site C Highway 29 Realignment Project
Farrell Creek Hydrotechnical Design Criteria (R1)
MEMORANDUM / TRANSMITTAL

TO: John
Highway Design Manager
R.F. Binnie

FROM: Des Goold, P.Eng. (NHC)

DATE: January 29, 2020

NO. PAGES: 9

CC: Gordon Wagner, P.Eng.
Bill Eisbrenner, P.Eng.
Project Manager
R.F. Binnie

PROJECT NO.: 3002853 (NHC)

REF. NO.: 1234567

RE: Site C Clean Energy Project
Engineering Design Services for Road and Bridge Infrastructure Works (Hwy 29)
W07 – Farrell Creek Hydrotechnical Design Criteria Rev 1

James,

The following hydrotechnical design criteria are proposed for the bridges:

1) Design Return Periods

   a. The following return periods are recommended in keeping with the BC Ministry of Transportation and Infrastructure’s (BC MOT’s) currently accepted probability of occurrence for extreme events:

      i. 10-year return period for hydraulic conditions during the construction period and reservoir filling period.

      ii. 50-year return period for wind speed that drives ice floes during normal reservoir operation (see 1b below).

      iii. 50-year return period for ice thickness during normal reservoir operation (see 1b below).

      iv. 10-year return period for wind speed that drives ice floes when reservoir levels are above or below the normal operating range.
v. 10-year return period for ice thickness when reservoir levels are above or below the normal operating range.

vi. 200-year return period for design stream flow and hydraulic conditions (flood levels, depths and velocities) during the period of reservoir operation.

vii. 200-year return period for wind-driven waves acting against shorelines during the period of reservoir operation.

viii. Overtopping of Highway 29, must not occur for (LGWs) having return periods of 1,000-years or less, during reservoir operation.

ix. 2,500-year return period for landslide generated wave (LGW) loads on bridge superstructure and substructure located above the normal operating range of the reservoir.

x. 10,000-year return period for landslide generated wave (LGW) loads on bridge piers founded within the reservoir.

b. The return period associated with ice loads is unique due the combination of independent probabilities for variables that define ice loads in reservoir environments. Two key variables are ice thickness and wind speed (which drives ice floes), and for this Project those have been assigned return periods of 50-years and 50-years, respectively. Combining just those two variable probabilities results in a return period of 2,500 years, were they to occur concurrently. If other variables, such as ice floe size and ice strength, are also assumed to be concurrent, the combined return period could be 2 to 4 time higher, or more.

2) Design floods estimation

a. The means of estimating the design floods on major tributaries shall be either of:

i. Single-station flood frequency analysis where suitable records exist; or

ii. Regional flood frequency analysis.

b. For small, ungauged watersheds, design floods will be estimated using procedures described in BC MOT’s Supplement to the TAC Geometric Design Guide (2019).

3) Climate Change Resilience

a. BC MOT’s Technical Circular T-04/19 shall be the governing procedural reference for incorporating climate change resilience into the Project.
b. BC Hydro’s Technical Data Report: *Climate Change Summary Report* (Volume 2, Appendix T of the Site C Clean Energy Project Environmental Impact Statement) shall be a governing technical reference for incorporating climate change resilience into the Project.

c. For mid- to large size watersheds (100s to 1,000s km²) like Farrell Creek, the Pacific Climate Impacts Consortium’s (PCIC’s) projections for selected Water Survey of Canada (WSC) gauge sites shall be used as a means of evaluating the potential impacts of climate change on streamflow.

d. The effects of future changes in temperature are not explicitly evaluated for hydrotechnical design. However, mean temperatures are expected to rise in the future, which will reduce the degree-days of freezing across the reservoir, resulting in thinner and weaker ice covers in the future.

4) Farrell Creek Realigned Channel Design Criteria

a. Design (10-year Annual Peak) Flow: 155 m³/s

b. Minimum Channel Bottom Width: 20 m

c. Minimum Freeboard (to Top of Berm): 0.3 m

d. Banks: Riprap armoured at 2H:1V

e. Low Flow Channel (Notch): 5 m Width x 0.3 m Depth

5) Reservoir Levels

a. The minimum normal reservoir level (MnNRL) will be El. 460.0 m.

b. The maximum normal reservoir level (MNRL) will be El. 461.8 m.

c. Emergency drawdown of the reservoir may occur as follows: from elevation 461.8 m to 452.5 m at 4.5 m per day, then to 446.5 or 444.0 m at 2.5 m per day (after BC Hydro Design Criteria for Rapid Drawdown, June 12, 2019; and the report: Site C Flood Discharge Facilities System Functionality and Conveyance Capabilities, 1016.Z.02.012.ENZ.00694.TM, July 2016").

6) Clearances

a. Structures (bridges and culverts > 3 m diameter) over non-navigable waterways shall have a minimum flood clearance of 1.5 m above the design flood level. Structures over navigable waterways shall have a minimum clearance as defined in Section 2.1.3 of the Structural Design Criteria prepared by WSP.
7) **Estimation of Degradation, Natural Scour, Contraction Scour, Local Scour and Total Scour**

   a. The estimation of all forms of scour and degradation shall be by methods consistent with the TAC Guide to Bridge Hydraulics, using the design return periods given in 1) a.

   b. Scour at culverts shall be assessed using procedures outlined in the US FHWA HDS 5 *Hydraulic Design of Highway Culverts*.

8) **Estimation of Aggradation Potential**

   a. Aggradation potential and its effect on design is being assessed on a site by site basis. No fixed criterion for aggradation has been established.

9) **Wave Prediction and Wave Loads on Bridge Substructure and Erosion Protection**

   a. Wind driven design wave height prediction shall be by use of the SWAN wave model developed at the Delft University of Technology, Netherlands.

   b. Statistical wind speeds will be determined by Extreme Value frequency analysis of hindcasted wind records at various BC Hydro meteorological stations within the Peace Valley. The historical wind record at Fort St. John will be used as a basis for hindcasting.

   c. Motorboat design wave height prediction shall be by methods described in *Wave Height from planning and semi‐planing small boats* (Maynord, 2005) and considering the Army Corps of Engineers Coastal Engineering Manual’s Table II-7-5.

   d. LGW wave propagation along the reservoir, run‐up and LGW forces acting on bridge structures shall be assessed by computational fluid dynamics (CFD) modelling using FLOW3D. LGW initial wave parameters will be provided by BGC Engineering Inc.

   e. LGW loads will not be considered in the design of erosion protection unless the LGW at specific locations has a low enough height and speed that will allow for stable protection design with a reasonable size of riprap (e.g. Class 2000-kg or smaller).

10) **Ice Loads**

   a. The determination of ice loads shall be in general accordance with the Canadian Highway Bridge Design Code S6-14 with the following exceptions:
i. Ice loads on the conical sections of piers will be determined according to the method provided by Ralston (1980) which provides a more rigorous 3D analysis of these types of loads.

ii. Impact loads on vertical structures may be reduced due to kinetic energy limitations of ice floe according to the methodology described in Sanderson (1980). The Code allowance for energy-limited impact loads applies to small ice floes and riverine ice processes, as opposed to lakes and reservoirs. In a lake or reservoir, the kinetic energy is limited by low velocities rather than small ice floe sizes, and the Sanderson approach allows for better quantification of the load reduction due to this effect.

iii. Thermal expansion loads will be determined according to the method developed by Carter (2001) which provides a more explicit method of determining thermal loads on structures based on both ice strength and ice thickness. Carter’s method accounts for additional factors with respect to the failure of ice sheets under thermal expansion and provides an explicit method to determine thermal expansion forces which is lacking in the Code.

iv. Ice loads on riprap (which the Code does not address) will be determined based on the experimental results by Sodhi (1996).

b. Ice thickness has been simulated for each year based on calibrated heat transfer coefficients and snow characteristics available through BC Hydro’s Physical Environment Team. Regional values for the heat transfer parameters will be obtained from calibration with lake ice thickness data from Cree Lake in northern Saskatchewan, which is the most representative, systematic ice growth data available in the region.

i. The estimated 50-year winter ice thickness is used for determination of thermal and uplift/ downdrag loads on bridge piers.

ii. The estimated 50-year spring ice thickness is used for determination of dynamic loads on bridge piers.

iii. As noted in 1a., 10-year ice thicknesses will be used when the reservoir is above or below the normal operating range.

c. The design ice floe width will be determined as the largest floe that would contact a single bridge pier; this width shall be determined as the maximum clear width between a set of three, adjacent piers at MNRL.

d. The design ice floe will be considered to act on bridge piers after reaching terminal velocity. Under normal reservoir operation, the terminal velocity of the design ice floe will be estimated using the 50-year hourly windspeed from the record at Fort St. John Airport (FSJ). When
reservoir levels are above or below the normal operating range, the 10-year (FSJ) windspeed will be used.

e. Ice strengths are as follows:

i. 1500 kPa for internally sound, mid winter ice used for developing loads on embankments and uplift/downdrag loads on piers (as applicable).

ii. For thermal loads developed in mid winter, the recommended flexural strength is 1,000 kPa.

iii. 700 kPa for ice floes present during spring break up, used for developing longitudinal forces acting on piers. The corresponding flexural strength would be 590 kPa.

iv. Where large spring floes interact with a shoreline over much larger scales – say the diameter of the floe – it is appropriate to adopt a lower crushing strength of 400 kPa. In this case, the corresponding flexural strength would be 340 kPa.

11) Ice Jams. It has been determined that ice jams do not pose a significant flood risk at any of the bridge sites; therefore, the spacing of piers at the bridges and the size of bridge waterways in general need not be constrained by need to limit the formation of jams.

12) Floating Debris

a. Where applicable, the assessment of floating debris and the potential loads associated with it will be determined based on the latest research published by the U.S. and international agencies, including but not limited to the following:

i. FHWA/IN/JTRP-2003/10 Debris Accumulation at Bridge Crossings; Laboratory and Field Studies.

ii. ERDC/CRREL TR-02-2 Maximum Impact Force of Wood Debris on Floodplain Structures.

13) Erosion Protection (Riprap)

a. Riprap used to protect the toe of fills, piers etc. during the construction period shall be sized to resist the greater of:

i. Maximum local stream velocities during the design flood;

ii. Plucking, stationary force transmission and dynamic impact forces from ice

b. Riprap protection used to protect the upper portion of fills during the service life of the bridges and roadways, after the reservoir has been filled, shall be sized to resist the greater of:
i. The impact force of the design wind-driven or boat wave;

ii. Plucking, stationary force transmission and dynamic impact forces from ice.

c. Riprap sizing for protection against stream velocities (per Item 12.a.i) shall be by methods outlined in the TAC Guide to Bridge Hydraulics, using the design floods determined by 2a) or 2b).

d. Riprap protection used to protect the inlet and outlet of culvert crossings shall be by methods outlined in TAC Guide to Bridge Hydraulics and/or US FHWA HDS 5 Hydraulic Design of Highway Culverts, as applicable.

e. Riprap sizing, gradation, thickness and revetment heights for protection against wind driven and motorboat wave impact (per Item 12.b.i) shall be by the Hudson Formula (or suitable equivalent) and wave run up estimation procedures outlined in the U.S. Army Corps of Engineer’s Manual Design of Coastal Revetments EM-1110-2-1614 (1995). A suitable equivalent to the Hudson Formula that may be used is the procedure developed by Melby (2005) outlined in the Coastal and Hydraulics Engineering Technical Note “Breakwater and Revetment Armour Stability”, ERDC/CHL CHETN-III-71 prepared for the US Army Engineer Research and Development Center.

f. Erosion protection and wind and boat waves will be provided where highway embankments overlap the Erosion Impact Line (EIL).

g. Erosion protection against wind and boat waves will be provided where bridge substructures overlap the Stability Impact Line (SIL).

h. Embankments and substructures that overlap the Wave Impact Line (WIL) may or may not be protected from erosion; the decision to protect will be based on acceptable levels of risk agreed to by BC Hydro and BC MOT at individual sites.

i. Erosion protection against wind and boat waves may be applied in other areas, including against exposed shale bedrock, where the design team considers there to be a high level of risk associated with erosion by waves and ice.

j. Where MoTI Riprap Classes are required, they shall have specifications that conform to the clauses and Table 205-A in Section 205 of the 2016 Ministry Standard Specifications for Highway Construction.

k. Riprap protected areas over granular soils, which are exposed to fluctuating reservoir levels and wave action, shall include an intermediate granular filter layer. Geotextile filters have a much shorter design life and should not be used because repeated cycling of reservoir levels, or rapid drawdown may cause them to fail prematurely.
l. The stability of highway fill material with respect to internal drainage will be addressed by the geotechnical design lead.

14) Alternative Forms of Erosion Protection

a. Alternative forms of erosion protection (i.e. other than riprap) may be used with the approval of BC Hydro and BC MOT, provided that those products are on the Recognized Products List (RPL).

b. Design methodologies for alternative protection shall be in accordance with supplier recommendations or recognized engineering standards for design of the product.

c. Erosion protection criteria 12a-b, f-i and k-l shall generally apply to alternative means of erosion protection, if used.

15) Waterway Hydraulic Opening Design (over streams)

a. Waterway opening design at bridges and culverts that cross stream channels will be in accordance with the TAC Guide to Bridge Hydraulics and will integrate all pertinent hydrotechnical design criteria contained in this document.
If you have any questions, please do not hesitate to contact me at 604.980.6011.

Sincerely,

northwest hydraulic consultants ltd.

Prepared by:
Graeme Vass, P.Eng.
Hydrotechnical Design Lead (W07)

Reviewed by:
Des Goold, P.Eng.
Principal
Hydrotechnical Design Manager (Highway 29)
APPENDIX B

100% Design Drawings 2184-281 to 283 and Suggested Realigned Channel T3 Reference Drawing
BERM (SEE GEOTEXTILE 2022+112.671)

LEGEND:

WATER LEVELS SHOWN CORRESPOND TO THE FOLLOWING FLOWS:

2. BASE MAPPING CREATED FROM LiDAR INFORMATION PROVIDED BY BC HYDRO IN AUGUST 2015 AND NORTHERN REGION.
3. CONTOUR INTERVALS ARE 1 m IN RIVER CHANNEL AND ON FLOODPLAIN;
4. NON-WOVEN GEOTEXTILE.
5. ALL PROFILES AND SECTION DETAILS SHOWN LOOKING NORTH.
6. RIPRAP SPECIFICATIONS AND INSTALLATION SHOULD CONFORM TO THE MINISTRY STANDARD.
7. Riprap shall be constructed in 5' high embankments placed in pairs of 5' left and 5' right.
8. Riprap base is constructed in 10' deep embankments placed in pairs of 10' left and 10' right.
9. SUGGESTED REALIGNED CHANNEL QUANTITIES TABLE

 channel quantities table

<table>
<thead>
<tr>
<th>item</th>
<th>description</th>
<th>quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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</tr>
<tr>
<td>2</td>
<td>Concrete</td>
<td>5000</td>
</tr>
<tr>
<td>3</td>
<td>Wood</td>
<td>100</td>
</tr>
</tbody>
</table>

MoTI 250-KG CLASS RIPRAP

BERM SHALL BE CONSTRUCTED WITH TYPE D MATERIAL PLACED AS PER SP 2.04.J.
APPENDIX C

Farrell Creek 1,000 and 10,000-yr LGW Impacts Memo (R2)
Dear Mr. Eisbrenner:

Northwest Hydraulic Consultants Ltd. (NHC) has conducted a computational fluid dynamics (CFD) modelling of the 1,000 and 10,000-year landslide-generated wave (LGW) propagation within the Site C reservoir at the Farrell Creek confluence. This document contains a summary of NHC’s CFD modelling results.

1 INTRODUCTION

1.1 General

When completed, the reservoir of the Site C Clean Energy Project will inundate parts of the existing British Columbia Halfway 29 running along the Peace River. As part of the Highway 29 Relocation, hydraulic assessments are being conducted in order to evaluate the feasibility of design options. The objective of NHC’s CFD modelling study was to assess the potential interaction of the LGW with certain design components proposed at key locations, where Highway 29 crosses Farrell Creek and Dry Creek (Figure 1). The scope of work included:

1. Develop a 3D CFD model of Farrell Creek crossing for three conditions: Initial embankment alignment (without bridge), revised alignment (without causeway and without bridge) and bridge structure (deck and piers);
2. Compute hydrodynamic forces at bridge piers.
1.2 Background

BGC\(^1\) developed a two-dimensional (2D) flow model to simulate the LGW propagation within the Site C reservoir, at both Lynx Creek and Farrell Creek. NHC conducted a preliminary review of BGC’s 2D LGW model results to assess the potential interaction of the LGW with certain design components proposed at key locations, where Highway 29 crosses Farrell Creek, Dry Creek and Lynx Creek (see Figure 1). Findings from the 2D LGW model review are summarized below.

BGC modelled potential LGWs propagation along the Peace River in the vicinity of the project sites using the depth-averaged flow model TELEMAC-2D. As shown in Figure 1, BGC identified three landslide cases (sites) near both Farrell Creek and Lynx Creek. The characteristics of BGC’s landslide cases for different return periods are shown in Table 1 for Lynx Creek and Table 2 for Farrell Creek. In both creeks, the 10,000-year LGW amplitudes are practically the same (15.0±0.2 m) for all landslide cases. This implies that, the proximity of the landslide case to the locations of interest is expected to have the most significant impact of the potential LGW impact at those locations. In both creeks, Landslide Case B is the closest to the future bridge location.

---

\(^1\) BGC has provided NHC with results of their study via several email correspondences between Oct. 18, 2018 and Oct. 24, 2018.
Table 1. Initial wave characteristics for landslide cases at Lynx Creek (BGC).

<table>
<thead>
<tr>
<th>Landslide Case</th>
<th>Return Period (years)</th>
<th></th>
<th></th>
<th></th>
</tr>
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<tr>
<td></td>
<td>10,000</td>
<td>2,500</td>
<td>1,000</td>
<td></td>
</tr>
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<td></td>
<td>Amplitude (m)</td>
<td>Period (s)</td>
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<td>Period (s)</td>
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<td>A</td>
<td>14.8</td>
<td>16.6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B</td>
<td>15.0</td>
<td>16.6</td>
<td>11.9</td>
<td>14.0</td>
</tr>
<tr>
<td>C</td>
<td>14.8</td>
<td>16.6</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 2. Initial wave characteristics for landslide cases at Farrell Creek (BGC).

| Landslide Case | Return Period (years) | | |
|---|---|---|
| | 10,000 | 1,000 | |
| | Amplitude (m) | Period (s) | Amplitude (m) | Period (s) |
| A | 14.8 | 15.2 | - | - |
| B | 15.2 | 15.2 | 9.6 | 11.3 |
| C | 14.8 | 15.2 | - | - |

2 CFD MODELLING

2.1 Model selection and validation

The CFD software selected for the Farrell Creek and Dry Creek LGW modelling is Flow-3D (Solver version 11.2.6), developed by Flow Science Inc. (www.Flow3D.com), which is the same software applied previously for the LGW modelling at Cache Creek2 and Halfway River3. Flow-3D is a CFD code widely used for advanced hydraulic application involving free surface flows. NHC has successfully applied and verified Flow-3D in several practical applications, including landslide-generated waves4,5,6.

---


2.2 Boundary conditions based on BGC’s LGW modelling

The LGW modelling conducted by BGC is described in BGC\(^7,8\). BGC used a hybrid modelling approach, in which wave generation parameters were estimated using Heller et al. (2009)\(^9\) empirical equations and wave propagation and runup were modelled numerically using the two-dimensional (2D) depth-averaged hydrodynamic model TELEMAC-2D\(^10\). In order to generate LGWs with the desired initial wave amplitude and wave period to match the predictions made by Heller et al. method, BGC imposed a squared sinusoidal wave at the landslide site as a boundary condition in their TELEMAC-2D model. It should be noted that because wave amplitude decays very quickly with distance, the wave amplitude imposed as boundary condition at the landslide site is higher than the corresponding “initial wave amplitude” reported by BGC (Table 1 and Table 2) at a certain distance away from the landslide site. To be consistent, NHC used in the Flow-3D CFD model the same wave boundary condition used by BGC in the TELEMAC-2D model. These wave boundary conditions are shown in Figure 2 for the 1,000-year and 10,000-year LGW’s created by landslide Case B at Farrell Creek.

The simulations at the Farrell Creek confluence were all conducted for the waves generated by landslide Case B, which is landslide location closest to the future bridge crossing. Furthermore, the LGW modelling was conducted with the Site C Reservoir at Maximum Normal Reservoir Level (MNRL) of El. 461.8 m.

![Figure 2. Wave boundary conditions used in the Flow-3D model; Landslide Case B at Farrell Creek.](image)

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2.3 Flow-3D model geometry

Using the bathymetric data from BGC’s TELEMAC-2D model plus the latest Lidar survey of ground topography, a Flow-3D model was generated encompassing an approximately 10 km long reach of the proposed Site C reservoir, and a 2.0 km reach of the corresponding Farrell Creek embayment (Figure 3 and Figure 4). The Flow-3D model uses a local coordinate system, in which the original UTM coordinates were translated and rotated such that the X-axis aligns with the landslide and the Y-axis is perpendicular to the landslide.

Several geometries were tested:

- The first set of simulations intended to assess the 1,000 and 10,000-year LGW propagation without the Farrell Creek bridge structure incorporating the embankment geometry following L2000A5P1 alignment\(^{11}\).
- The second series of simulations were conducted to study the 10,000-year LGW at the location of the future Farrell Creek bridge. This simulation was run with highway alignment L2000A5P1 (same as the first series) but the causeway was removed. The results of this simulation were intended to assist the bridge design team with the potential impact of the 10,000-year LGW on a long bridge option. No bridge was included in this simulation.
- The third set of simulations included the proposed 6-span bridge structure\(^{12}\) at the Farrell Creek mouth in order to evaluate the 10,000-year LGW impact on the bridge (e.g. forces on piers). For the bridge modelling simulations, the revised highway alignment DL2000A2P8 alignment\(^{13}\) was incorporated in the model.

An elevation view of the bridge geometry is shown in Figure 5 and a schematic view as simulated in Flow-3D is shown in Figure 6.

All CFD model surfaces, including existing ground and bathymetry, bridge structure and highway embankment are simulated as rigid and non-erodible solids. Deformations to the surfaces due to LGW impact (e.g. scouring) are not accounted for.

---

\(^{11}\) NHC received the finished surface geometry via email on November 14, 2018.

\(^{12}\) The bridge geometry used in the simulation was based on the design received from WSP on July 29\(^{th}\), 2019

\(^{13}\) The highway embankment DL2000A2P8 was received from Binnie on July 29\(^{th}\), 2019.
Figure 3. Overall view of the Flow-3D model of the Future Site C Reservoir at Farrell Creek and Dry Creek (L2000A5P1 alignment, Nov. 2018).

Figure 4. Schematic view of the Flow-3D model of the Future Site C Reservoir at Farrell Creek at existing conditions (left) and reservoir conditions (right) both with highway embankment L2000A5P1.

Figure 5. Elevation view of the Farrell Creek bridge looking north (WSP, July 2019).
2.4 Computational Mesh

The Flow-3D numerical mesh is comprised of several mesh blocks of different resolutions (i.e. cell sizes). The mesh cells in the area of main wave propagation, between Farrell Creek and the landslide location, were 5 m cubes. In the vicinity of the bridge, a mesh size of 0.5 m was used. The total number of cells in all mesh blocks combined was over 10 million.

3 WAVE MODELLING RESULTS: WITHOUT FARRELL CREEK BRIDGE

3.1 1,000 and 10,000-year LGW Propagation with Causeway

*Figure 7* illustrates how the 1,000-year LGW propagates within the Future Site C Reservoir at 10, 20, 30, 50, 75, and 80 seconds. The wave reaches the Farrell Creek mouth (i.e. the location of the future bridge) at approximately 70 seconds (*Figure 7*). The 1,000-year LGW does not overtop the embankment.

The 10,000-year LGW propagation within the Future Site C Reservoir is shown in *Figure 8* at 10, 20, 30, 50, 70, and 80 seconds. After about 25 seconds, the front of the LGW reaches the opposite north bank and runs up overland. The remaining of the LGW moving north-west reaches the Farrell Creek mouth (i.e. the location of the future bridge) at approximately 65 seconds (*Figure 8*). The 10,000-year LGW does not overtop the embankment.

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14 All reported times are referenced (zeroed) to the start of the LGW shown in *Figure 2*.
Figure 7. Snapshots of the 1,000-year LGW propagation within the Future Site C Reservoir at Farrell Creek at 10, 20, 30, 50, 75 and 80 s (red colour indicates surface velocity ≥ 10 m/s).
Figure 8. Snapshots of the 10,000-year LGW propagation within the Future Site C Reservoir at Farrell Creek at 10, 20, 30, 50, 70 and 80 s (red colour indicates surface velocity ≥ 10 m/s).

An envelope of the maximum water surface elevation as the LGW passes the proposed bridge section is shown in Figure 9 for the 1,000 and 10,000-year waves. The maximum water surface elevations along the bridge location (i.e. at Farrell Creek mouth) are 465.1 m and 471.1 m for the 1,000 and 10,000-year waves, respectively. As shown in Table 3, the corresponding maximum wave amplitudes at the creek mouth are 3.3 m and 9.3 m for the 1,000 and 10,000-year waves, respectively.
As the LGW impacts the embankment and travels through the creek mouth, the maximum wave velocities reach approximately 10.0 m/s and 13.0 m/s for the 1,000 and 10,000-year waves, respectively (see Table 3).

<table>
<thead>
<tr>
<th>Location</th>
<th>Return Period (years)</th>
<th>LGW Height (m)</th>
<th>LGW Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Farrell Creek Mouth</td>
<td>10,000</td>
<td>9.3</td>
<td>13.0</td>
</tr>
<tr>
<td></td>
<td>1,000</td>
<td>3.3</td>
<td>10.0</td>
</tr>
<tr>
<td>Dry Creek Mouth</td>
<td>10,000</td>
<td>2.1</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>1,000</td>
<td>1.3</td>
<td>2.0</td>
</tr>
</tbody>
</table>

As the LGW continues to propagate within the Future Site C reservoir, the wave reaches the Dry Creek mouth after approximately 235 seconds for the 1,000-year wave and 225 seconds for the 10,000-year wave. Screenshots of the LGW’s at the Dry Creek mouth are shown in Figure 10 and summarized in Table 3. As shown in this figure, the velocities at this location are approximately 2.0 m/s and 3.0 m/s for the 1,000 and 10,000-year waves, respectively. Furthermore, the wave amplitudes at this location are 1.3 m and 2.1 m for the 1,000 and 10,000-year waves, respectively (see Table 3). As shown in Figure 11 (right), velocities of the 10,000-year LGW along the embankment reach approximately 4.0 m/s to the east of Dry Creek.
Figure 10. Snapshots of the 1,000-year LGW within the Future Site C Reservoir approaching Dry Creek at 235 s; coloured by water surface elevation (left) and velocity (right).

Figure 11. Snapshots of the 10,000-year LGW within the Future Site C Reservoir approaching Dry Creek at 225 s; coloured by water surface elevation (left) and surface velocity (right).

As shown in Figure 12, by the time the 1,000 and 10,000-year waves approach the Lynx Creek East embankment, the wave velocity is reduced to approximately 1.0 m/s and 2.0 m/s, respectively. However, along the embankment and at the MNRL, areas with larger velocities of almost 3.0 m/s are observed when the 10,000-year LGW impacts the embankment (see Figure 12 right). However, it is likely that the impact of the LGW generated by the Lynx Creek landslide Case C reaching this location; i.e. along the embankment at Lynx Creek East will be more intense. Hence, modelling of the LGW generated by that landslide is required in order to evaluate the LGW impact on the embankment at Lynx Creek East region.
3.1.1 Interaction of LGW with Embankment at Farrell Creek

Screenshots of the LGW impacting the embankment (i.e. causeway) and propagating at the creek mouth are shown in Figure 13 and Figure 14 for the 1,000-year waves at 70 s and 75 s, respectively. The 1,000-year LGW runs up the embankment reaching a maximum elevation of El. 470.0 m (see Figure 13 left). The minimum water level (i.e. wave rundown) is approximately El. 457.0 m; i.e. approximately 4.8 m below MNRL. This corresponds to a change of about 13 m in water level along the south face of the embankment. This drop in water level occurs in about 5 seconds. A section view of the LGW runup is shown in Figure 15. The maximum LGW velocities reaching a maximum value of approximately 10.0 m/s (see Figure 13 and Figure 14).

Figure 13. Snapshots of the 1,000-year LGW within the Future Site C Reservoir at Farrell Creek impacting the embankment at 70 s; coloured by water surface elevation (left) and velocity (right).
Figure 14. Snapshots of the 1,000-year LGW within the Future Site C Reservoir at Farrell Creek impacting the embankment at 75 s; coloured by water surface elevation (left) and surface velocity (right).

Figure 15. Section view (looking west) of maximum runup of the 1,000-year LGW impacting on the east embankment (St. 2001+200) at Farrell Creek at 69 s coloured by velocity (top) and water surface elevation (bottom).

Screenshots of the LGW impacting the embankment (i.e. causeway) and propagating at the creek mouth are shown in Figure 16 and Figure 17 for the 10,000-year waves at 64 s and 73 s, respectively. The maximum wave runup along the embankment occurs around St. 2001+200 m and the water surface raised to El. 475.0 m. The minimum water level (i.e. wave rundown) at this location is approximately El. 455.0 m; i.e. approximately 6.8 m below MNRL. This corresponds to a change of about 20 m in water level along the south face of the embankment. This drop in water level occurs in about 5 seconds. A section view of the 10,000-year LGW runup at this location is shown in Figure 18. At 64 s, the maximum wave velocity along the embankment occurs farther west (at St. 2001+120 m) as shown in Figure 16 right.
Figure 16. Snapshots of the 10,000-year LGW within the Future Site C Reservoir at Farrell Creek impacting the embankment at 64 s; coloured by water surface elevation (left) and surface velocity (right).

Figure 17. Snapshots of the 10,000-year LGW within the Future Site C Reservoir at Farrell Creek impacting the embankment at 73 s; coloured by water surface elevation (left) and surface velocity (right).
Pressure maps due to the LGW impact on the embankments are shown in Figure 19 and Figure 20 for the 1,000 and 10,000-year LGW. The portions of the embankment that are impacted by the 1,000-year LGW experience pressures ranging from 25 kPa (in the upper portions of the runup) to 50 kPa (at MNRL). Corresponding values for the 10,000-year LGW are within a range from 25 kPa (in the upper portions of the runup) to 75 kPa (at MNRL). Notice that the model provides the pressure caused by the LGW on the external face of the embankment, but interstitial pore pressure within the riprap could be different. It should also be noted that the colour maps shown in Figure 19 and Figure 20 refer to the instantaneous values of pressure acting along the south face of the embankment, and do not account for the negative pressure due to the relatively rapid drop in water level from maximum (i.e. wave runup) to minimum.
(i.e. wave rundown).

**Time = 69.0 s**

Site C MNRL

**Time = 73.0 s**

Site C MNRL

---

**Figure 19.** Pressure map of the 1,000-year LGW impact on the east embankment (i.e. causeway) at Farrell Creek at 69 s and 73 s coloured by pressure (view looking north).

---

**Time = 63.0 s**

Site C MNRL

**Time = 66.0 s**

Site C MNRL

---

**Figure 20.** Pressure map of the 10,000-year LGW impact on the east embankment (i.e. causeway) at Farrell Creek at 63 s and 66 s coloured by pressure (view looking north).
3.2 10,000-year LGW Propagation without Causeway

In order to assess the LGW pattern at the location of a long bridge option without a causeway, the model was run by removing the extension of the embankment (L2000A5P1) within the reservoir. This exercise was conducted to provide the bridge design team with insight on potential impact of the LGW on the bridge superstructure. A section view showing the maximum wave height under the bridge is shown in Figure 21 with and without the causeway. As shown in this figure, by eliminating the causeway, the maximum wave height stays almost the same as the case with the causeway. The maximum LGW surface elevation is approximately EL. 471.2 m, observed along the east end of the bridge location.

![Figure 21. Envelope of maximum water surface elevation for the 10,000-year LGW with and without the causeway at Farrell Creek (view looking north).](image)

Screenshots of the LGW propagating within the reservoir at the Farrell Creek bridge location are shown in Figure 22 and Figure 23 at 70 s and 73 s, respectively. The LGW propagates within the reservoir at the Farrell Creek mouth in a northwest direction. As shown in Figure 22, the local surface velocity of the LGW at the location of the bridge reaches almost 16.0 m/s. Furthermore, the LGW impacts the right bank (at the east end of the creek mouth), and runs up the existing ground. This results in the LGW height reaching approximately 475.0 m. Following the wave impact on the bank, a weaker secondary wave forms and travels within the reservoir in southwest direction (see Figure 23).
Figure 22. Snapshots of the 10,000-year LGW within the Future Site C Reservoir at the Farrell Creek bridge location at 70 s; coloured by water surface elevation (top) and surface velocity (bottom).

Figure 23. Snapshots of the 10,000-year LGW within the Future Site C Reservoir at the Farrell Creek bridge location at 73 s; coloured by water surface elevation (top) and surface velocity (bottom).
4 WAVE MODELLING RESULTS: WITH FARRELL CREEK BRIDGE

4.1 10,000-year LGW Propagation with Bridge

Following the preliminary modelling of the LGW propagation within the reservoir, the 10,000-year LGW was simulated with the bridge installed. The objective of this modelling effort was to evaluate the interaction of the LGW with the bridge including the forces exerted on the bridge piers and superstructure.

A section view of the maximum LGW elevation under the Farrell Creek bridge is shown in Figure 24. As shown in this figure, the wave surface elevation reaches a maximum of about 472.9 m at the west end of the bridge. That is, the LGW does not impact the bridge abutments or superstructure (i.e. deck or girders).

![Figure 24. Envelope of maximum water surface elevation for the 10,000-year LGW at the Farrell Creek bridge location (view looking north).](image)

Propagation of the 10,000-year LGW within the reservoir is shown in Figure 25 and Figure 26 at 70 s and 77 s, respectively. The LGW pattern is, in general, similar to that observed in the simulation without the bridge and with the causeway removed (see Figure 22 and Figure 23). The LGW travels in a northwest direction and has a maximum local velocity of about 16.0 m/s. After the LGW impacts the right bank (adjacent to Pier 5), it is reflected back towards the bridge. As a result, a secondary wave impact in the southwest direction is observed at the bridge piers. As shown in Figure 26, when the LGW impacts Pier 2 (at about 77 s), the secondary wave impacts Pier 4.
Figure 25. Snapshots of the 10,000-year LGW within the Future Site C Reservoir at the Farrell Creek bridge location at 70 s; coloured by water surface elevation (top) and surface velocity (bottom).

Figure 26. Snapshots of the 10,000-year LGW within the Future Site C Reservoir at the Farrell Creek bridge location at 77 s; coloured by water surface elevation (top) and surface velocity (bottom).
4.2 Hydrodynamic Forces

Hydrodynamic forces at the piers are caused mainly by pressure forces. Pressure forces at the pier are due to the pressure differential between opposite sides of the pier caused by the passage of the LGW. The differences in water surface elevations across the pier and the flow separation behind it cause a pressure differential across the pier. This pressure differential multiplied by the projected vertical area of the pier generates a horizontal pressure force. Pier forces were only assessed for the 10,000-year LGW.

At each pier, Flow-3D integrates the pressure acting on the pier surface to compute the three spatial components of pressure force \( F_x, F_y, F_z \) acting on the pier. The horizontal components \( F_x \) and \( F_y \) acting on a pier are illustrated in Figure 27. \( F_x \) acts parallel to the bridge axis, being positive towards the east abutment, while \( F_y \) acts perpendicular to the bridge axis, being positive towards the north and into Farrell Creek. The net horizontal force \( F_h \) is the vector sum of \( F_x \) and \( F_y \). The angle of attack is \( \alpha = 0 \) if \( F_h \) acts perpendicular to the bridge axis in a north direction.

\[
F_x : \text{Horizontal pressure force along bridge axis, positive towards the east (kN)}
\]
\[
F_y : \text{Horizontal pressure force perpendicular to bridge axis, positive towards the north (kN)}
\]
\[
F_h = \left( (F_x)^2 + (F_y)^2 \right)^{1/2} : \text{Net horizontal pressure force (kN)}
\]
\[
\alpha : \text{angle of attack (degrees)}
\]

**Figure 27. Sketch defining horizontal pressure forces at pier (plan view).**

A summary of the peak horizontal forces acting on the bridge piers is shown in Table 4. As shown in this table, the 10,000-year LGW first impacts Pier 4 at about 70.3 s. As the LGW travels in a northwest direction, Pier 1 is impacted last at about 79.4 s. The interaction of the LGW with Pier 5 occurs as the wave runs up the ground and impact this pier almost perpendicular to the bridge axis. The maximum force magnitude of approximately 1,839 kN is observed at Pier 2.
Table 4. Summary of peak horizontal forces exerted by the 10,000-year LGW as predicted by Flow-3D at bridge piers.

<table>
<thead>
<tr>
<th>Pier</th>
<th>Time(^a) (s)</th>
<th>F(_x)(^b) (kN)</th>
<th>F(_y)(^b) (kN)</th>
<th>F(_h)(^b) (kN)</th>
<th>(\alpha)(^b) (deg)</th>
</tr>
</thead>
<tbody>
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<td>Pier 1</td>
<td>79.4</td>
<td>-651</td>
<td>112</td>
<td>661</td>
<td>-80</td>
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<tr>
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<td>75.2</td>
<td>-1,819</td>
<td>272</td>
<td>1,839</td>
<td>-82</td>
</tr>
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<td>Pier 5</td>
<td>71.2</td>
<td>90</td>
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<td>1,162</td>
<td>4</td>
</tr>
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NOTES:

a) All reported times are referenced (zeroed) to the start of the LGW shown in Figure 2.
b) Bridge piers are numbered in a west-to-east direction.
c) Refer to Figure 27 for a definition sketch of force components and direction.

The time evolution of the horizontal pressure force at the bridge piers is shown in Figure 28. As shown in this figure, the horizontal force reaches a maximum just after the arrival of the LGW at each pier. The peak is followed by a secondary rise in the force magnitude as the secondary wave (i.e. reflected wave from the east bank) reaches the piers. This is more pronounced at Piers 3 and 4. The magnitude of the force exerted by the secondary wave on Piers 3 and 4 is, in general, 30% to 40% of the peak for that pier.

![Figure 28. Time evolution of the horizontal pressure force (F\(_h\)) at bridge piers as predicted by Flow-3D.](image)

The pressure load distribution at the time of the peak horizontal force on each pier is shown Figure 29. As shown in this figure, the maximum pressure load is observed at Pier 5 with the magnitude reaching almost 500 kN/m.
5 SUMMARY AND CONCLUSIONS

5.1 1,000 and 10,000-year LGW Modelling Results without Bridge

- Based on date from BGC, Landslide Case B was selected for CFD modelling of LGW impact on Farrell Creek. Landslide Case B is located closest to the Farrell Creek mouth, and also has the largest initial wave amplitude compared to Cases A and C.

- The 1,000 and 10,000-year LGW’s arrive at the Farrell Creek mouth respectively about 70 s and 65 s after the landslide is triggered.

- Modelling both 1,000 and 10,000-year LGW’s at Farrell Creek indicated that the LGWs do not overtop the proposed causeway (embankment design L2000A5P1 and DL2000A2P8).

- The maximum wave heights at the Farrell Creek mouth (i.e. at the location of the proposed bridge) are approximately 3.3 m and 9.3 m for the 1,000 and 10,000-year LGWs, respectively.

- The maximum local velocities along the south and west slopes (i.e. faces) of the embankment (i.e. causeway) are approximately 10.0 m/s and 13.0 m/s for the 1,000 and 10,000-year LGW’s, respectively. These relatively large velocities appear to develop as the LGW impacts the embankment.

Figure 29. Peak pressure load distributions on the bridge piers.
The LGW runup along the causeway reaches maximum elevations of El. 470.0 m and 475.0 m for the 1,000 and 10,000-year LGW’s, respectively. The LGW rundown along the causeway reaches minimum elevations of El. 457.0 m and 455.0 m for the 1,000 and 10,000-year LGW’s, respectively.

The maximum pressure due to the LGW impact on the embankment is approximately 75 kPa at MNRL (this does not include the negative pressure due to the relatively rapid water level drop from runup to rundown).

The 1,000 and 10,000-year LGW’s arrive at the Dry Creek mouth respectively about 235 s and 225 s after the landslide.

At Dry Creek, the wave heights are approximately 1.3 m and 2.1 m for the 1,000 and 10,000-year LGW’s, respectively.

The LGW velocities at Dry Creek are 2.0 m/s and 3.0 m/s for the 1,000 and 10,000-year LGW’s, respectively.

The current results of the LGW modelling at Lynx Creek East – East and Lynx Creek East – West are not comprehensive. Modelling of the Lynx landslide Case C will be required in order to evaluate the potential LGW impact along the embankment at Lynx Creek East – East and Lynx Creek East – West.

5.2 10,000-year LGW Modelling Results with Bridge

The maximum 10,000-year LGW height under the Farrell Creek bridge is approximately 11.1 m, corresponding to a water surface elevation of 472.9 m.

The local wave velocities at the location of the Farrell Creek bridge reaches approximately 16.0 m/s.

The 10,000-year LGW does not impact the bridge abutments, deck or girders (i.e. does not overtop the bridge).

The 10,000-year LGW impacts Pier 4 of the proposed bridge design first at about 70.3 s after the landslide.

The maximum horizontal force occurs at Pier 2 with a magnitude of approximately 1,840 kN at 76 s.

Following the LGW impact on the right bank at Farrell Creek mouth (i.e. east end of bridge), the wave reflects back into the reservoir and impacts the piers. This secondary wave is weaker than the initial impact and has a different orientation. The magnitude of the force by the secondary wave is 30% to 40% of the peak force. The initial impact has a northwest orientation but the secondary reflected wave travels in the southwest direction.

The largest peak pressure load is observed at Pier 5 with a magnitude of about 500 kN/m.
Sincerely,

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ENCLOSURE

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