# PARSNIP RIVER BRIDGE NO. 1185
# JOHN HART HIGHWAY 97
# HYDROTECHNICAL DESIGN BRIEF FOR BRIDGE REPLACEMENT

## FINAL SUBMISSION

Prepared for:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

BC Ministry of Transportation and Infrastructure
Northern Region – Prince George, BC

17 July 2017

NHC Ref. No. 3001953.1004

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Prepared by:

**Northwest Hydraulic Consultants Ltd.**
North Vancouver, BC


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# 1 INTRODUCTION

The BC Ministry of Transportation and Infrastructure (MoTI) is proposing to replace Parsnip River Bridge No. 1185, which carries Highway 97 over the Parsnip River 140 km north of Prince George, BC. MoTI intends to replace the existing three-span bridge as it has reached the end of its service life.

Northwest Hydraulic Consultants Ltd. (NHC) has been retained by MoTI (under Contract No. 356CS0874) to prepare this hydrotechnical design brief in support of the detailed design for the replacement bridge, which is being carried out by CWMM Consulting Engineers Ltd. (CWMM).

In addition, NHC has also prepared and submitted the following reports to MoTI:

- "Parsnip River Bridge No. 1185 John Hart Highway 97 Hydrotechnical Assessment and Design Constraints for Bridge Replacement" (Phase I Report) (Final Report; NHC, 2016a).

- "Parsnip Bridge Replacement Project – Phase II Climate Change Assessment Update (Draft Report; NHC, 2016b).

## 1.1 Scope of the Detailed Hydrotechnical Design

The scope of the hydrotechnical design for replacement of the Parsnip River Bridge is as follows:

- Liaise with the Design Team to develop bridge concepts and provide input regarding span arrangements, foundation options, scour potential, hydraulic clearances, ice and stream loads, etc.
- Carry out analyses of climate change effects in the watershed to ensure the increase in design discharge for climate change recommended during Phase I is appropriate.
- Address other hydrotechnical issues, as required, related to geotechnical and structural engineering, roadway geometrics, environmental and other regulatory approvals.
- Provide preliminary and detailed hydrotechnical design services for the preferred bridge replacement option; includes refinement of scour estimates and ice loading and design of erosion and scour protection work as required.
- Review the bridge design and drawings at each deliverable milestone to ensure consistency with hydrotechnical design and assist with preparation of contract documents where related to hydrotechnical design.
- Act as Engineer of Record for hydrotechnical component of the project; prepare and compile all documentation related to these services.
- Provide construction phase services related to hydraulic issues such as answer contractor queries, review of any proposed in-stream temporary works required during construction, and carry out site inspections.
- Coordinate development and assembly of post-project documentation, including record reports and drawings, BMIS data sheets, SER report updates, etc.

## 1.2 Design Codes and References

The following design codes and reference documents form the basis of the hydrotechnical design:

Design Codes:

- CAN/CSA-S6-14 in conjunction with the BC MoT Supplement to CHBDC S6-14
- BC MoTI Supplement to TAC Geometric Design Guide (2007)

Ministry Standards and Guidelines:

- BC MoTI Standard Specifications for Highway Construction (2016)

Hydrotechnical Design Guidelines:

- TAC Guide to bridge Hydraulics (2004)
- Hydraulic Design of Stable Flood Control Channels (NHC, 1984)
- Professional Practice Guidelines – Legislated Flood Assessments in a Changing Climate in BC (V1.1) (APEGBC, 2012)

## 1.3 Proposed Layout of the New Bridge

Refer to CWMM Consulting Engineers Ltd. (CWMM) Drawing 1185-101 (CWMM, 2017) for the proposed general arrangement of the replacement bridge.

# 2 HYDROTECHNICAL DESIGN PARAMETERS

## 2.1 Mean flood flow and Normal High Water Level

The normal high water level at the upstream face of the existing bridge, associated with the mean flood of 945 $m^3/s$, is El. 685.8 m.

## 2.2 Design Flow

The design flows for the new bridge are the 100-year and 200-year annual peak (instantaneous) flows. The 100-year peak flow is used to establish the minimum clearance required for navigation, in accordance with the Navigable Waters Protection Act (NWPA). The 200-year peak flow is used for most other aspects of hydrotechnical analysis and design (e.g. scour estimates and riprap protection works).

Streamflow data from Water Survey of Canada (WSC) Gauge No. 07EE007, *Parsnip River above Misinchinka River*, which is located approximately 7 km upstream of Bridge No. 1185, was used for the frequency analysis. The return period estimates were then scaled up for the bridge based on the difference in drainage areas. The estimated 100-year and 200-year peak flows at the bridge are 1,506 $m^3/s$ and 1,588 $m^3/s$ respectively (NHC, 2016a).

During Phase I of the hydrotechnical design and for hydrotechnical design purposes the 200-year peak flow estimate had been increased by 23 percent, to 1,953 $m^3/s$, accounting for the potential effects of future climate change (NHC, 2016a). This percent increase was determined using the Pacific Climate Impacts Consortium's (PCIC) simulation of the Peace River basin with input of stationary properties.

Subsequently, MoTI requested that an extreme value analysis be conducted of the simulation taking account of non-stationarity in the future streamflow projections by PCIC. A report was prepared describing the non-stationary assessment and recommending that a revised percent increase of 33.7 percent applied to the 200-year peak flow (NHC, 2016b). However, it has since been determined that the percent increase was determine incorrectly in that report. NHC's revised recommendation for percent increase based on the non-stationary simulation is 12.6 percent increase. In this case the 200-year peak flow increases from its current, estimated value of 1,588 $m^3/s$ to 1,788 $m^3/s$. A revised report for the non-stationary analysis is included in Appendix A of this design brief.

## 2.3 Design Flood Levels and Velocities

A summary of design levels, average velocities and maximum velocities are shown in Table 2.1

**Table 2.1 Summary of the estimated design flood levels and velocities**

| Design Flow | Peak Discharge $m^3/s$ | Upstream Flood Level[1] El., m | Avg Velocity m/s |
|---|---|---|---|
| 100-year | 1506 | 687.5 | 2.1 |
| 200-year | 1588 | 687.7 | 2.2 |
| 200-year+12.6 percent (Future 200-yr) | 1788 | 688.1 | 2.3 |

**Notes:**

1. Flood levels are at the upstream face of the existing bridge.
2. Average velocities are the maximum computed values within the hydraulic opening.

## 2.4 Design High Ice Level

Please refer to NHC's Phase I Report (NHC, 2016a) for complete details. Winter ice affected water levels were evaluated for late-winter solid ice cover conditions. Water levels and discharges for late winter solid ice covers were obtained from late winter discharge measurements and a theoretical ice jam rating curve was generated from this data – the curve was then calibrated and shifted to the bridge site. From this data a Monte Carlo simulation was carried out to determine the frequency distribution of maximum, annual water level during ice breakup. The 200-yr, maximum breakup water level, or 'Design High Ice Level" is estimated to be El. 689.3 m.

## 2.5 Minimum Recommended Bridge Clearance

The future 200-year, and the current, 100-year maximum instantaneous flood levels at the existing bridge are estimate to be roughly El. 688.1 m, and El. 687.5 m respectively. The high ice elevation is estimated as El. 689.3 m, which is above both of these open water flood levels. A freeboard or clearance of 1.5 m above the high ice elevation is recommended to account for variability in the ice surface in an ice jam and therefore, the minimum soffit elevation should be at least El. 690.8 m (NHC, 2016a). With the bridge soffit set to this elevation, both Navigable Water Protection Act (NWPA) and MoTI Standard clearance requirements are met by default.

## 2.6 Recommended Waterway Opening for Replacement Bridge

The waterway for the proposed replacement bridge has a bottom width of 135.2 m, on square at a reference El. 682.4 m, which roughly corresponds to a mean annual flow in the river. When projected upward at slopes of 2H:1V, the waterway has a width of approximately 148 m at the level corresponding to a 2-year flood, El. 685.8 m (NHC, 2016a). This closely matches the existing bridge waterway opening.

## 2.7 Replacement Bridge Layout

CWMM have designed the proposed replacement bridge as a three span steel plate girder bridge with a total span of 184 m perpendicular to the existing river banks. The three spans, from east to west, are 56 m (east span), 72 m (main span), and 56 m (west span). The bridge will be located roughly 20 m upstream of the existing bridge with the abutments and piers of the proposed bridge located at roughly the same lateral location as the existing bridge. The proposed bridge has two piers, each a pile group comprised of five slightly inclined 1200 mm dia. steel pipe piles with concrete diaphragms between each pile. The pile penetration depth at the piers is anticipated to be El. 661.2 m. The abutments consists of a concrete retaining wall and pile cap with eleven 610 mm dia. steel pipe piles spaced in two rows with the waterside row inclined towards the centre of the channel. The anticipated pile penetration depth for the both abutments is El. 675.2 m. The proposed layout and details of the bridge are provided in CWMM drawings 1185-100 to 1185-106 (CWMM, 2017). The proposed configuration and layout of abutment riprap is discussed in Section 2.10.

## 2.8 Ice Loads on Bridge Piers

General advice with respect to ice loading is provided in NHC's Phase I Report (NHC, 2016a). The 200-year ice thickness is estimated as 1.0 m and the recommended ice strength for use in calculating loads is 1,100 kPa. All ice forces should be applied at the high ice level, El. 689.3 m.

### 2.8.1 Longitudinal loads on piers

Longitudinal ice loads will occur against the leading piles of the two bridge piers, which are located within the main channel and in-line with the principal direction of flow and ice movement. These ice loads are a function of the pier width, nose shape and inclination angle, as well as the strength and thickness of the ice. For a 1220 mm diameter pier, the crushing load is 3,073 kN. The 79 degree inclination of the leading pile results means that the ratio of bending load to crushing load likely exceeds 3.0, so the ice will fail by crushing long before bending. Longitudinal loads are applied parallel to the long axis of the pier. Ice loads acting transverse to this long axis should also be applied in combination with the longitudinal loads. Two cases must be considered. In one case, a transverse load of 15% of the longitudinal load should be applied in combination with 100% of the longitudinal load. In the other case, a transverse load that is a function of nose shape should be applied with 50% of the longitudinal load. For the round pier nose in CWMM's design, the transverse load would be 34% of the longitudinal load.

### 2.8.2 Vertical (Uplift) loads on piers

The maximum value of vertical ice load acting on a pier will likely be between 1,500 kN and 1,600 kN, depending on the adopted pier geometry. In the case of CWMM's pier design, the vertical load can be distributed evenly to all five piles, or 300 to 320 kN/ pile.

### 2.8.3 Static (Thermal) loads

Static, unit ice loads as high as 260 KN/m may be applied transversely to a pier if ice occurs on only one side of the pier. These should be treated as point loads.

## 2.9 Long Term Bed Elevation Changes and Scour

Despite low sediment transport capacity in the channel, historical bathymetric surveys show that cycles of sediment buildup (aggradation) and removal (degradation) have occurred at the existing bridge with the average bed levels ranging from El. 682.0 m to El. 680.0 m. The current, average bed level at the proposed bridge is El. 681.6 m and the minimum (thalweg) level is El. 681.2 m. For the purposes of bridge design, it would be prudent to allow for aggradation or degradation depths of at least 1 m relative to current minimum bed levels, a range of El. 680.2 m to El. 682.2 m. Scour depth estimates should be made relative to El. 680.2 m, assuming that is the lowest starting bed level when scour first starts to occur.

### 2.9.1 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 endfills, abutments, piers, etc. At the existing bridge crossing the channel width is relatively consistent with the upstream channel width and the bridge abutments are well set back from the active channel with only limited constriction caused by the bridge piers.

The 200-year contraction scour elevation has been estimated using the FHWA HEC-18 procedure (Richardson and Davis 2001; TAC 2004) with the following inputs.

- Design flow = 1788 $m^3/s$ (future 200-year peak flow)

- 200-year hydraulic depth through the bridge =5.9 m
- Channel top width at bridge waterway opening = 155 m
- Channel top width of approach channel = 177 m
- Bed material transport exponent = 0.59
- 200-year flood level at bridge = El. 688.1 m

This results in a scoured depth due to contraction of 6.4 m below the 200-year flood level, or El. 681.7 m. The current average bed elevation at the proposed bridge is about El. 681.6 m, so there is negligible contraction scour potential.

### 2.9.2 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 concentrations of flow such as the confluence of two or more channels.

At Highway 97 the river is straight, laterally stable and the Misinchinka confluence is too distant to contribute to scour; therefore, the only relevant trigger for natural scour here would be the occurrence of an unusually large flood. The 200-year natural scour elevation has been estimated using the Modified Blench procedure (TAC 2004) with the following inputs:

- Channel top width at approach and bridge waterway opening = 155 m
- Design flow = 1788 $m^3/s$ (future 200-year peak flow)
- Local channel slope upstream of bridge = 0.0006 m/m
- Z factor = 1.5
- $D_{50}$ = 35 mm
- 200-year flood level at bridge = El. 688.1 m

The estimated natural scour depth at the existing bridge opening is 6.8 m below the 200-year flood level, or 681.3 m, suggesting there is minor scour potential that could extend up to 0.3 m below the average bed elevation.

### 2.9.3 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. Both proposed bridge piers are supported by 1220 mm diameter steel pipe piles. The ambient bed level surrounding the pier, accounting for degradation and natural scour is El. 679.9 m. The estimated local scour depth at the bridge piers is 2.2 m below the ambient bed level, El. 677.7 m.

## 2.10 Riprap Protection

The channel at the existing and proposed bridge locations is laterally stable with minor bank erosion occurring along the west bank. The west bridge abutment is located well back from the top of the existing, roughly 8 m high bank with little risk of being undermined due to erosion by the river.

The east abutment is supported on bridge end fill projecting upwards from the gently sloping, existing bank. To provide protection to the east bridge end fill, a layer of 700 mm thick, MoTI 100 kg Class riprap is recommended to be placed along the end fill sloping upwards at 2H:1V from El. 683.2 m to El. 688.3 m with a Class I non-woven geotextile (Nilex 4512 or equivalent) between the riprap and fill. To provide protection to the east bridge end fill toe a second layer of 700 mm thick, MoTI 100 kg Class riprap is recommended to be placed along the end fill toe on the previously placed riprap layer, sloping upwards at 2H:1V from El. 683.2 m to the 2-year water surface elevation, El. 685.8 m as shown in CWMM drawing number 1185-101 (CWMM, 2017). The two layer thickness along the end fill toe provides sufficient rock volume to launch in the event the riprap protection becomes undermined over the life of the bridge.

There has historically been erosion of the bank due to local runoff from the bridge. Local runoff should be tight-lined down the slopes below the east and west abutments to avoid such erosion problems in the future.

If the piers cannot tolerate scour down to the design local scour level, El. 677.7 m, then a riprap apron should be provided around the base of each pier. To be effective, the aprons would have to be 10 m wide, centred on the long axis of each pile group and extending 3 m beyond the ends of the leading and downstream piles. This apron width assumes the riprap is placed at the existing streambed elevation and allows for settlement to occur if the river bed degraded in the future. The aprons should be constructed using minimum 50 kg riprap at a placement thickness of 660 mm (2x $D_{50}$). However, if temporary berms/ causeways are constructed to allow for pier construction, the scour apron riprap may need to be increased in size up to 100-kg (and 830 mm thickness) to accommodate an increase in local velocities around piers due to the berm obstructing the flow.

# 3 REFERENCES


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# APPENDIX A

Parsnip Bridge Replacement Project – Phase II Climate Change Assessment Update

NHC Ref. No. 3001953.100

06 December 2016

**R.F. Binnie & Associates Ltd.**
678 Vancouver Street,
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V2L 2P3

**Attention:** **Gordon Wagner, P.Eng**
Project Manager

**Via email:** gwagner@binnie.com

**Re:** **Parsnip Bridge Replacement Project – Phase II**
**Climate Change Assessment Update (Revised)**

## Introduction

Northwest Hydraulic Consultants Ltd. (NHC) prepared and submitted a hydrotechnical assessment and design constraints report for the replacement of Parsnip River Bridge No. 1185 in March 2016 (NHC 2016). That study was completed for the BC Ministry of Transportation and Infrastructure (MoTI) under Contract 356 CS 0901. The report presented design water levels, flood levels, minimum waterway opening dimensions, scour and riprap estimates for the Parnsip River Bridge Replacement Project. It also included the results of an assessment of the effect that future climate change may have on the design flow and hydraulics at the bridge.

## Overview of the effects of Climate Change on Streamflow

Potential changes in peak flows due to climate change are a product of two competing processes. Higher rainfall intensities are expected to generally increase in Northeast BC as the climate changes, and this result provides insight as to how climate change could affect future flooding in smaller watersheds where the largest annual floods have historically been driven by rainfall. On the other hand, warmer temperatures are likely to result in lower snow accumulation, thus potentially lowering the magnitude of future snow-driven peak flows, which tend to occur in the larger watersheds like Parsnip River. The use of a process based hydrologic model is the most appropriate method to assess how climate change may affect flooding caused by both rainfall and snowmelt. While this type of hydrologic modelling is outside the scope of this particular report, model output from other groups has been made public for the region.

## Available Data

Detailed assessment of potential changes in peak flows over time is made possible via streamflow projections produced by the Pacific Climate Impacts Consortium (PCIC) for gauged sites across BC. PCIC used the Variable Infiltration Capacity (VIC) hydrologic model (Liang et al. 1994). Model output is available online[1], and is supplied as an ensemble of 23 daily streamflow time series' from 1955 to 2100. The model was driven by downscaled climate model output for 23 combinations of Global Circulation Models (GCMs) and carbon production scenarios from the AR4 IPCC climate assessment report (IPCC 2007). PCIC downscaled the climate model output to approximately 300 arc-second grids using Bias Corrected Spatial Disaggregation methods (Werner 2011) in order to supply daily maximum and minimum temperature, and daily precipitation to the VIC model. PCIC supplies output for the B1 (conservative), A1B (moderate), and A2 (severe) future $CO_2$ scenarios[2]. Though lower emissions scenarios are modelled and output is produced, most engineering design focuses on the higher $CO_2$ scenarios, as up to this point, there is no suggestion that mankind is progressing on anything other than the 'business as usual' production levels of $CO_2$.

PCIC has applied the VIC model and GCM ensembles to a number of sites in the Peace region, including the Water Survey of Canada (WSC) Gauge: *Parsnip River above Misinchinka River* (08EE007).

## Scope of this report

The climate change assessment carried out for the March 2016 report involved conducting flood frequency analyses on the 23 projected time series for Parsnip River. The climate change analysis associated with this report was performed on the projected time-series <u>with assumed stationary properties</u>.

Upon their review of NHC's March 2016 report, MoTI requested that the flood frequency analyses of PCIC's projected time series be conducted <u>with non-stationary properties.</u> The requested update is the subject of this report.

## Flood frequency analysis of projected streamflow data

Flood frequency analysis was performed on the projected annual maximum daily flow time series for each site using the 'extRemes' package for R (Gilleland and Katz 2014)[3]. The GEV distribution was fit using the Generalized Maximum Likelihood Estimation (GMLE) methodology (Martins and Stedinger 2000) with the distribution fit using L-moments. The GEV distribution has been found to provide the

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[1] Pacific Climate Impacts Consortium, University of Victoria, (Jan. 2014). Station Hydrologic Model Output. Downloaded from https://www.pacificclimate.org/data/station-hydrologic-model-output on: August 25, 2016

[2] https://www.ipcc.ch/publications_and_data/ar4/wg1/en/spmsspm-projections-of.html

[3] The projected time series provided by PCIC do not include annual peak instantaneous flow time series

best fit to historic data at WSC 06EE007 and it is necessary to have a consistent distribution choice so that the analysis of change could be applied in the same manner for all projected time series analyses.

The ratio between historical maximum daily annual flow and historical maximum instantaneous annual flow was assessed for WSC Gauge 08EE007. This ratio was used as a peaking factor to post-process model output from maximum daily average to instantaneous peak flow. Because no other option was available, it is assumed that the relationship between instantaneous and maximum annual daily flow remains constant over time and into the future.

### Non-stationary flood frequency analysis

Flood frequency analysis under the conditions of climate change requires further analysis to determine whether flood quantiles may be expected to change over time (i.e. whether they are non-stationary). Many potential options for accounting for this non-stationarity of flood quantiles are reviewed in Khaliq et al. (2006). The GEV distribution has three parameters (location, scale, and shape). In a non-stationary analysis, the location and scale parameters are allowed to vary as functions of time, while the shape parameter is left constant (e.g. Kharin and Zwiers 2005; Gilroy and McCuen 2012; El Adlouni et al. 2007).

Parameter optimization for the GEV distribution with time as a covariate has previously been done in multiple different ways. Gilroy and McCuen (2012) and Kharin and Zwiers (2005) used a rolling window (of 51 years in both cases) to calculate a time series of static location and scale parameters (fit using maximum likelihood estimation), then fit models to these parameter time series to estimate time varying quantiles. El Adlouni et al. (2007) extended this methodology with the Generalized Maximum Likelihood Estimation (GMLE) method for non-stationary extreme value analysis. In this method, the likelihood estimator is maximized for the entire series in multi-dimensional space, and location and scale parameters are represented directly as functions of time. This eliminates the need to choose a window size for the rolling parameter estimation.

NHC used the GMLE methods from El Adlouni et al. (2007), implemented within the 'extRemes' package to estimate linear parameter functions (for the location and scale parameters) with time as a covariate. Time was considered as an index, ranging from 1 to 145 (representing 1955 to 2100) as in Gilroy and McCuen (2012). This non-stationary estimation method was applied to each of the 23 GCM ensemble members produced by PCIC for WSC Gauge 08EE007. Diagnostic plots, illustrating the distribution fit (in transformed space), were produced for each of the ensemble members and each site. The slope of the location and scale functions (fitted to the data via GMLE methods) indicates how severely the flood quantiles are thought to vary over time for each ensemble member.

### Blending of PCIC model output with observations

For the historic period (1950 – present) the model output presented by PCIC is driven by historic observations. The flood quantiles estimated for the historic time series are used to represent current conditions and are the base point for comparison to potential future changes. NHC used the PCIC model output to supply the slope of the functions representing the GEV location and scale parameters. The shape parameter from the WSC observations was maintained as a constant over time. The modelled slope of the location and scale functions was applied to the WSC observed GEV location and scale parameters to estimate potential changes in quantiles of the observed maximum daily series over time. In essence, the location and scale parameters from the observations represent the intercept (where Time = 0, on the last year in which observations are available) of the parameters, and the slopes are obtained from the model output. Ensembles of peak flows from the PCIC model output are shown in Figure 1, and illustrate the range of potential trends in the peak flow series, dependent on GCM and $CO_2$ scenario. Note the inflection point near the present day, where model time series' transition from historic observed to future predictions.

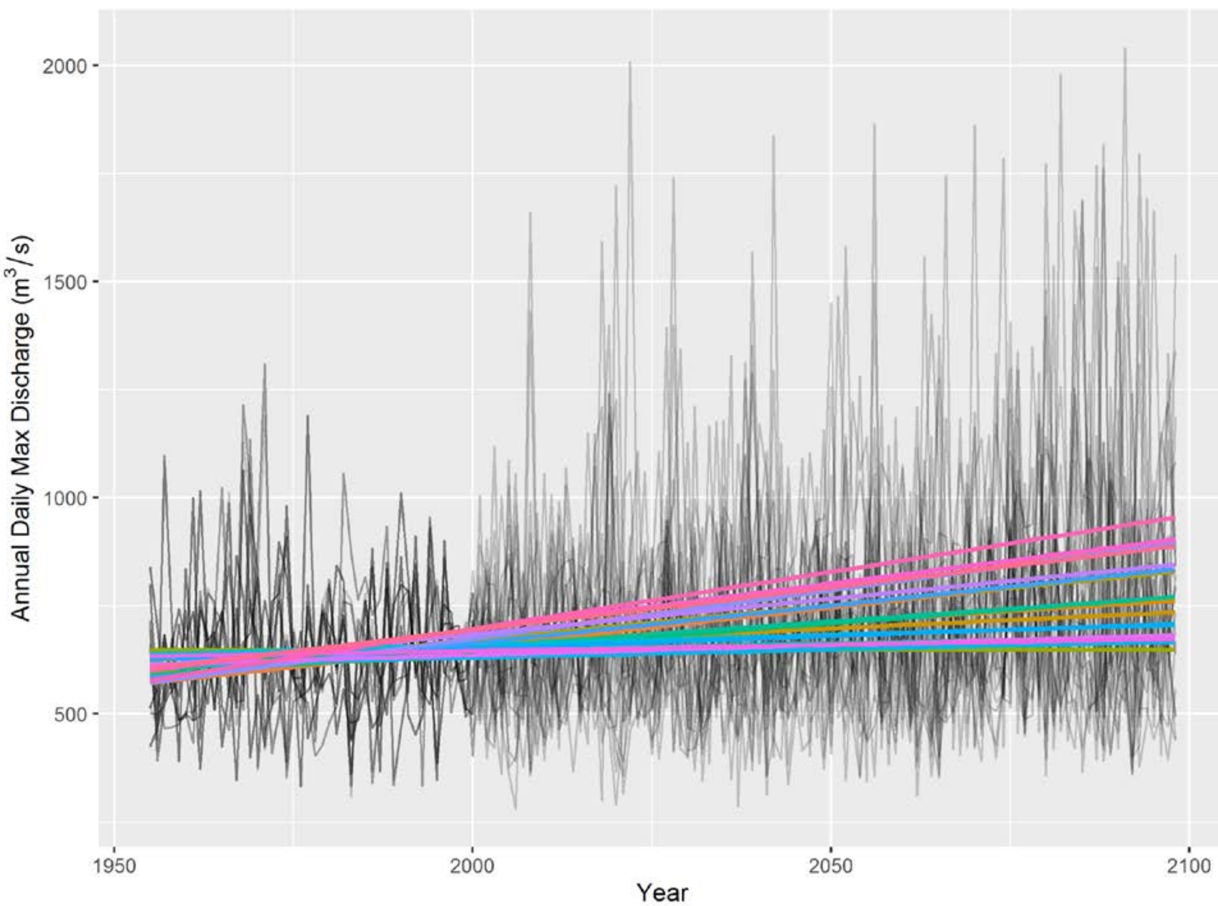


*Figure 1 Ensemble output of maximum annual daily flow for the PCIC projected time series at WSC Gauge 07EE007. Linear trends are shown separately for each of the 23 ensemble time series.*

As a final post-processing step, the instantaneous-daily peaking factor for the historic record at WSC 08EE007 (factor [slope] = 1.03; Figure 2) was applied to all values of projected annual daily maximum flow quantiles to produce estimates of maximum instantaneous flow quantiles ($m^3/s$).

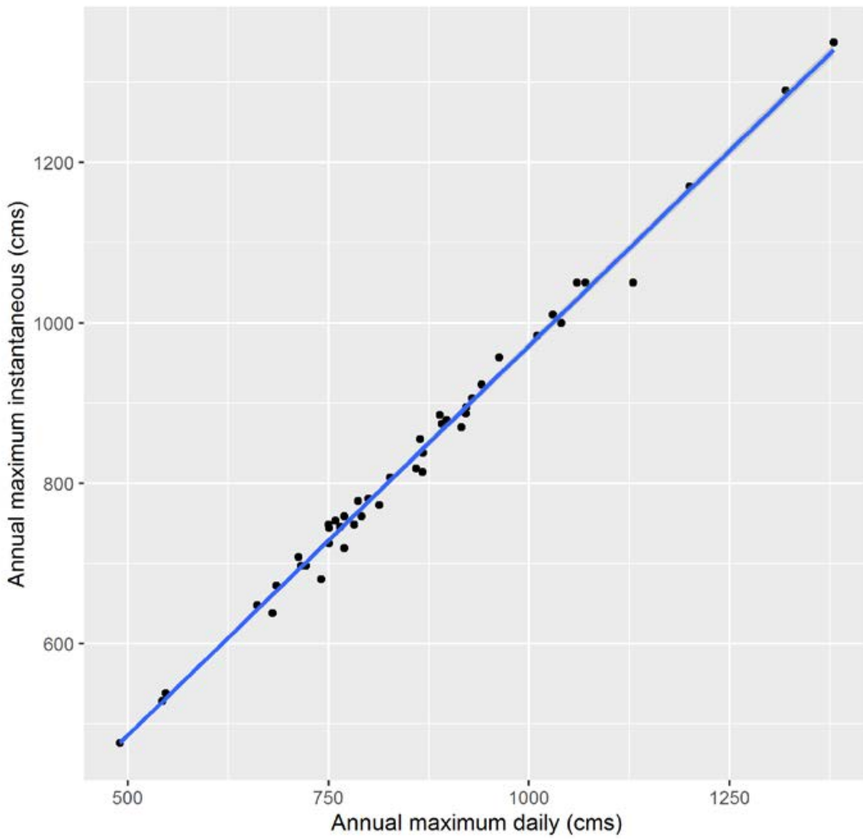


*Figure 2 Correlation between maximum annual daily flow and instantaneous annual maximum flow for observed floods at WSC 07EE007. A linear model (with 0 intercept) is shown in blue; the slope is 1.03.*

## Results

Ensemble model output has been summarized in Figure 4 via stratifying ensemble time series by the $CO_2$ scenario and calculating summary statistics (minimum, 25th percentile, median, mean, 75th percentile, and maximum) for each year. Results show that the ensemble median is expected to increase for all scenarios and return periods. As would be expected, the higher $CO_2$ scenarios show greater increases than the low scenario (B1). Additionally, the ensemble spread increases over time. Changes from baseline (present day quantiles from PCIC simulation of the historic record) to median, projected end of century quantiles are summarized in Table 1.

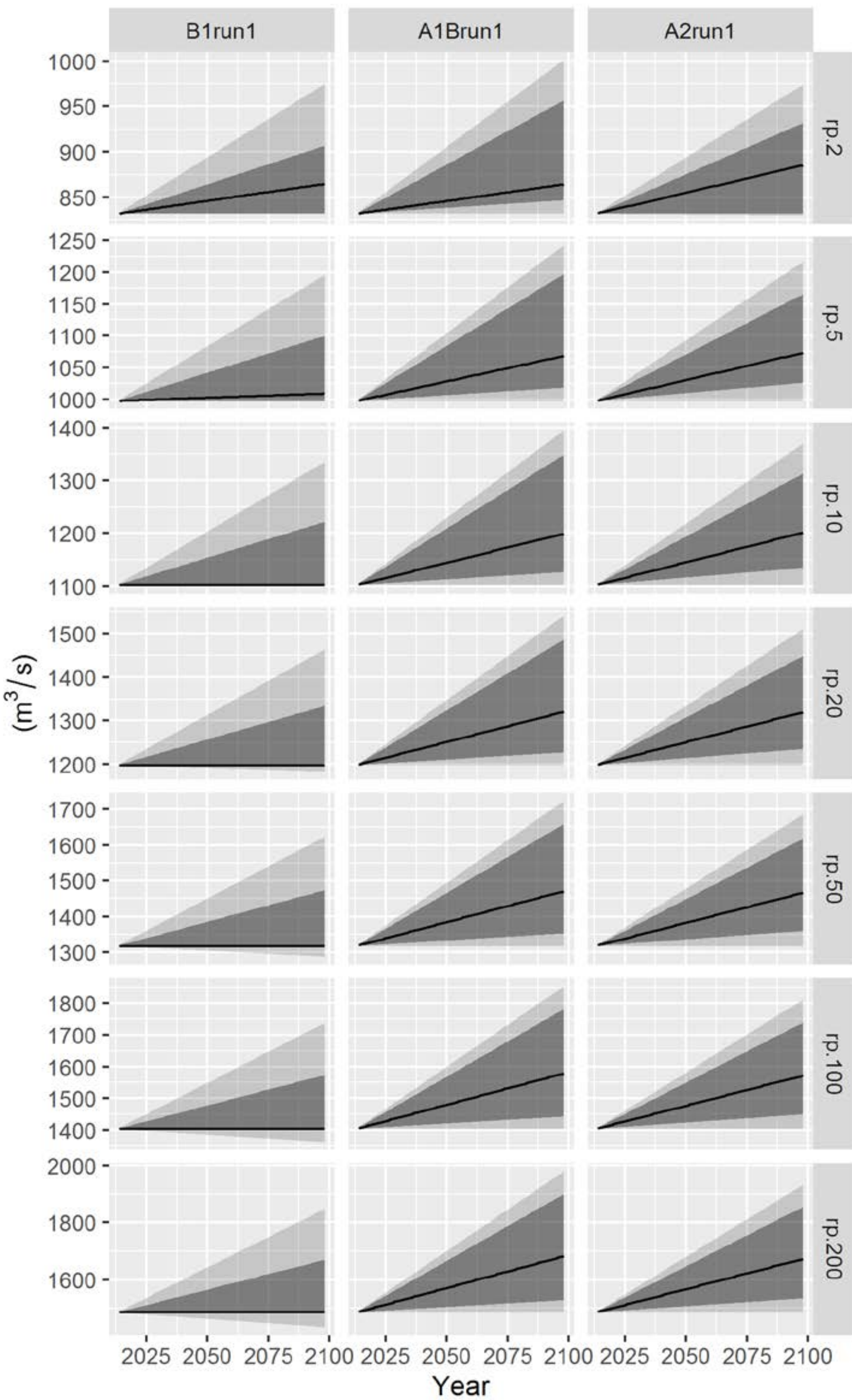


*Figure 3 Changes in instantaneous peak flows from present to end of century for Gauge 07EE007. Plot columns represent $CO_2$ scenarios, and plot rows represent return periods. Dark lines in each plot indicate median, dark grey bands indicate 25th-75th percentile, and light gray bands indicate minimum and maximum.*

*Table 1 Summary of expected ensemble median changes for WSC 07EE007 between Baseline (historical observed present day) and end of century (EOC) of instantaneous peak flows.*

| RP | Baseline ($m^3/s$) | A1B–EOC ($m^3/s$) | A1B % change | A2–EOC ($m^3/s$) | A2 % change |
|---|---|---|---|---|---|
| 2 | 832 | 864 | 3.9 | 885 | 6.4 |
| 5 | 997 | 1068 | 7.2 | 1,073 | 7.6 |
| 10 | 1101 | 1,200 | 8.9 | 1,202 | 9.1 |
| 20 | 1198 | 1,321 | 10.3 | 1,320 | 10.2 |
| 50 | 1317 | 1,471 | 11.7 | 1,467 | 11.3 |
| 100 | 1403 | 1,579 | 12.5 | 1,572 | 12.0 |
| 200 | 1486 | 1,683 | 13.3 | 1,673 | 12.6 |

The median percentiles of the projected 200 year instantaneous flow for both modelled scenarios are within 10 $m^3/s$ of one another, and essentially equal. NHC recommends using the more conservative and realistic carbon model scenario (A2run1) 200 year instantaneous flow increase of 12.6 percent for future design purposes.

The estimated 200-year peak instantaneous flow at the Parsnip River Hwy 97 Bridge is 1,588 $m^3/s$ (NHC 2016), and applying a 12.6 percent increase to that results in a future, end-of-century 200-year peak instantaneous flow of 1,788 $m^3/s$. This results in the 200-year design water surface elevation increasing by 0.4 m from El. 687.7 to El. 688.1 and the 200-year mean velocity increasing by 0.1 m/s from 2.2 m/s to 2.3[4] m/s. Of note is that the design high ice level at the bridge (El. 689.3 m) is still the governing factor in determining the design elevation of the bridge soffit.

*    *    *    *    *

[4] Average velocities are the maximum computed values within the hydraulic opening.

We trust this draft report meets with your expectations. Please let us know if you have any comments or questions regarding its contents.

Sincerely,

**Northwest Hydraulic Consultants Ltd.**

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