POST-EA INFORMATION PACKAGE INCLUDING
AN UPDATED PROJECT DESCRIPTION
ALL SEASON ROAD TO PRAIRIE CREEK MINE

APPENDIX 1-8

SUBMITTED IN SUPPORT OF:
Water Licences MV/PC2014L8-0006, and
Land Use Permits MV/PC2014F0013

SUBMITTED TO:
Mackenzie Valley Land and Water Board
Yellowknife, NT X1A 2N7
Parks Canada,
Nahanni National Park Reserve
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SUBMITTED BY:
Canadian Zinc Corporation
Vancouver, BC, V6B 4N9

February 2019
1.0 INTRODUCTION

Tetra Tech Canada Inc (Tetra Tech) was retained by NorZinc Ltd./Canadian Zinc Corporation (CZN) to update prior Tetra Tech preliminary assessments of design water levels and velocities at major watercourses where bridge crossings are proposed along the Prairie Creek Mine access road. The prior assessments are documented in reports dated November 19, 2012, December 18, 2014, and June 6, 2016.

The present work provides design water levels and velocities for 10 bridge crossing sites for: (1) existing conditions 2-year and 100-year flows based on hydraulic models reflecting field survey data collected in 2017 and 2018 and (2) future-conditions flows at three bridge crossing sites where encroachments are proposed within the 100-year floodplain.

Site plans with survey information used to develop the hydraulic models was provided by Allnorth “issued for review” drawings dated October 2018, in pdf and AutoCAD formats that included the underlying spot elevation survey data. The surveys were completed in August 2017 and August 2018.

A list of the assessed crossings is presented below; Kilometer Post (KP) identifiers are as identified on the Allnorth 2018 site plans. Because of design phase alterations to the road alignment, the present KP identifiers and crossing locations may differ from those used in previous reports.

1. KP 6.0 Casket Creek. Site survey completed 2018/08/22.
2. KP 13.3 Funeral Creek. Site survey completed 2017/08/17.
4. KP 23.3 Sundog Creek. Site survey completed 2017/08/17.
5. KP 25.4 Sundog Tributary. Site survey completed 2017/08/16.
6. KP 42.9 Sundog Tributary. Site survey completed 2018/08/25.
8. KP 87 Old Road at Tetcela Trib. Site survey completed 2018/08/16.
9. KP 89.5 Old Road at Tetcela Mainstem. Site survey completed 2018/08/14.
In addition to the above sites, a check was made of the crossing of the Sundog Creek tributary at KP 39.2, located at its confluence with the main Sundog Creek floodplain. This crossing was previously assessed as KP 39.4 (2014 alignment) in Tetra Tech’s December 18, 2014 report, Stream Crossing Design Water Levels for Prairie Creek Mine Access Road. After discussion with Allnorth, it was decided that further analysis was not necessary for this crossing at present.

The hydraulic modeling results presented herein were initially provided to CZN in November 2018 to support then-concurrent bridge crossing design work by Allnorth Consultants.

### 2.0 DESIGN APPROACH

Existing conditions design water levels and velocities were determined with HEC-RAS modelling of open water 2-year and 100-year peak flows. Model cross sections were extracted from the Allnorth site survey data in proximity to the field survey section locations with relatively high densities of spot elevations to define the channel geometry, including bathymetry.

Site survey data provided by Allnorth was extended using CZN high-resolution LiDAR elevation data to obtain the downstream channel slope and other required information beyond the extents of the ground survey. The LiDAR data were obtained (flown) in June 2012 with 15 cm vertical accuracy and horizontal point density at around one point per square metre. The Allnorth site plans also used this LiDAR-derived information to supplement the ground survey data.

Estimates of Manning “n” hydraulic roughness values for the channel and floodplain areas were typically the same values used in prior preliminary assessments of the crossing sites, based on prior review CZN photographs which included aerial reconnaissance images taken on various dates and ground photographs taken by Hatfield Consultants during site surveys in late September 2014. The “n” values, especially for new or altered crossing locations due to road realignments, were further assessed based on Allnorth ground survey photographs showing channel and overbank conditions.

Allnorth’s bridge design approach included avoidance of any encroachment within the 2-year inundation limits identified by the hydraulic modelling for existing conditions or, alternatively, the bankfull (ordinary high-water mark) condition determined by Allnorth field observations. Additional hydraulic modelling of proposed conditions was performed for bridge crossing sites where abutments and/or approach fill were proposed within the 100-year inundation limits. The results of the additional modelling were then used by Allnorth to complete the designs.

The water levels provided herein are for open water flow conditions based on hydraulic modelling. Allnorth is applying a freeboard as a factor of safety for model uncertainty as well as an allowance for ice and debris. As recommended previously, the possibility of ice-influenced high water levels should be investigated by field observations at breakup when ice influences will be at their greatest. The design high water elevations and/or freeboard amounts may need to be adjusted if breakup period field observations identify significant ice effects.
3.0 DESIGN DISCHARGES

Tributary basin areas for each of the crossing locations were determined from digital analysis of GeoBase 1:50,000 scale Canadian Digital Elevation Data derived from the National Topographic Data Base. The watershed analysis was done using Global Mapper software, with a visual inspection of the delineation results against 1:50,000 scale Toporama mapping to confirm that the results were reasonable and to make adjustments as necessary.

100-year peak flows were estimated with the same regional analysis equation previously used for the 2012 and 2014 studies. The equation to determine the 100-year discharge is given below, with discharge Q in cubic metres per second (m³/s) and watershed area A in square kilometers (km²).

\[ Q_{100} = 1.888 \ A^{0.751} \]

2-year peak flows, representing ordinary high water or bankfull conditions, were estimated from log-normal frequency analyses results for the same regional stations used previously, yielding the equation below.

\[ Q_2 = 0.2333 \ A^{0.8996} \]

4.0 DESIGN FLOWS AND WATER LEVELS

The results of the hydrologic and hydraulic analyses for each of the road crossings are summarized and presented in the Table and Figures sections at the end of this report. Site photographs are included in an additional section.

Table 1 presents a summary of the basin areas, design flows, channel slope, and Manning’s n roughness coefficients for each stream crossing.

Table 2 summarizes the results of the hydraulic modelling of existing conditions for 2-year and 100-year peak flows for the model section representing the crossing centreline. Generally, the results are for a surveyed (and modelled) section immediately upstream of the centreline location. Water surface top width, elevation, and velocities are tabulated, together with maximum water depth measured at the channel thalweg (the deepest part of the channel).

Table 3 summarizes the results of the hydraulic modelling of proposed encroachment conditions for 100-year peak flows for the model cross section immediately upstream of the bridge. This modeling was done for three crossings with significant road fill through the 100-year floodplain areas: Polje Creek at KP 53.2, Tetcela Tributary at KP 87.0, and Tetcela Mainstem at KP89.5. The information provided for these crossings consists of the water surface elevation, the energy grade line, and the channel mean velocity. The energy grade line elevation represents the surface level of slack water that is ponded against the upstream side of the road embankment.

Figures are provided for each crossing to show the following, as applicable:

1. Site plan view showing HEC-RAS model cross sections and site survey topography on background 2012 orthophoto image;
2. Existing condition model results showing the near-centreline channel cross section with water levels;
3. Proposed condition model bridge section showing the proposed bridge abutments and road fill; and
4. Proposed condition model 100-yr model results showing the water levels and energy grade line for the cross section immediately upstream of the proposed bridge.

Unless stated otherwise, all cross sections are plotted to be viewing downstream.
Photos are provided for each crossing to show channel characteristics including vegetation in overbank and floodplain areas. The tables, figures, and photos are presented in lieu of separate report sections for each crossing which would duplicate or otherwise present the same information.

5.0 LIMITATIONS OF REPORT

This report and its contents are intended for the sole use of NorZinc Ltd./(Canadian Zinc Corporation) and their agents. Tetra Tech Canada Inc. (operating as Tetra Tech) does not accept any responsibility for the accuracy of any of the data, the analysis, or the recommendations contained or referenced in the report when the report is used or relied upon by any Party other than NorZinc Ltd./(Canadian Zinc Corporation), or for any Project other than the proposed development at the subject site. Any such unauthorized use of this report is at the sole risk of the user.
6.0 CLOSURE

We trust this report meets your present requirements. If you have any questions, please contact the undersigned.

Respectfully submitted,
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Attachments:  Tables (3)
              Figures (27)
              Photos (20)
              Appendix A – Tetra Tech’s Limitations on the use of this Document
TABLES

Table 1 - Summary of Design Flows and Hydraulic Parameters
Table 2 - Hydraulic Model Results near Bridge Centreline for Existing Conditions
Table 3 - Hydraulic Model Results with Proposed Encroachments
<table>
<thead>
<tr>
<th>2018 Road KP</th>
<th>Stream Name</th>
<th>Basin Area (km²)</th>
<th>Q2 (m³/s)</th>
<th>Q100 (m³/s)</th>
<th>Slope (m/m)</th>
<th>Channel</th>
<th>Overbank</th>
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<tr>
<td>6</td>
<td>Casket Creek</td>
<td>15.2</td>
<td>2.70</td>
<td>14.6</td>
<td>0.031</td>
<td>0.055</td>
<td>0.055</td>
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<td>13.3</td>
<td>Funeral Creek</td>
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<td>0.34</td>
<td>2.6</td>
<td>0.108</td>
<td>0.055</td>
<td>0.070</td>
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<td>20.3</td>
<td>Sundog Tributary</td>
<td>6.1</td>
<td>1.2</td>
<td>7.3</td>
<td>0.06</td>
<td>0.055</td>
<td>0.07</td>
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<td>23.3</td>
<td>Sundog Creek</td>
<td>14.5</td>
<td>2.6</td>
<td>14.1</td>
<td>0.037</td>
<td>0.055</td>
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<td>25.4</td>
<td>Sundog Tributary</td>
<td>5.8</td>
<td>1.1</td>
<td>7.1</td>
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<td>42.9</td>
<td>Sundog Tributary</td>
<td>17.9</td>
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<td>16.5</td>
<td>0.0075</td>
<td>0.04</td>
<td>0.08</td>
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<td>53.1 &amp; 53.2</td>
<td>Polje Creek &amp; Tributary</td>
<td>102</td>
<td>15</td>
<td>61</td>
<td>0.005</td>
<td>0.03</td>
<td>0.2</td>
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<td>87</td>
<td>Tetcela Tributary</td>
<td>92.5</td>
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<td>57</td>
<td>0.0065</td>
<td>0.06</td>
<td>0.2</td>
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<td>89.5</td>
<td>Tetcela Main Channel</td>
<td>727</td>
<td>88</td>
<td>266</td>
<td>0.004</td>
<td>0.03</td>
<td>0.2</td>
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<td>121</td>
<td>Grainger River</td>
<td>41.5</td>
<td>6.7</td>
<td>31</td>
<td>0.022</td>
<td>0.04</td>
<td>0.2</td>
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### Table 2 - Hydraulic Model Results near Bridge Centreline for Existing Conditions

<table>
<thead>
<tr>
<th>2018 Road KP</th>
<th>Stream Name</th>
<th>Q2 Peak Flow</th>
<th>Q100 Peak Flow</th>
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<td>Top Width (m)</td>
<td>Elevation (m)</td>
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<td>13.3</td>
<td>Funeral Creek</td>
<td>5.75</td>
<td>922.46</td>
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<td>20.3</td>
<td>Sundog Tributary</td>
<td>9.74</td>
<td>1293.30</td>
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<td>23.3</td>
<td>Sundog Creek</td>
<td>10.15</td>
<td>1128.45</td>
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<td>25.4</td>
<td>Sundog Tributary</td>
<td>1.9</td>
<td>1069.71</td>
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<td>42.9</td>
<td>Sundog Tributary</td>
<td>20.29</td>
<td>829.38</td>
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<td>53.2</td>
<td>Polje Creek</td>
<td>18.37</td>
<td>708.92</td>
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<td>87</td>
<td>Tetcela Tributary</td>
<td>17.03</td>
<td>295.04</td>
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<td>89.5</td>
<td>Tetcela Main Channel</td>
<td>102.91</td>
<td>269.66</td>
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<tr>
<td>121</td>
<td>Grainger River</td>
<td>32.15</td>
<td>521.32</td>
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### Table 3 - Hydraulic Model Results with Proposed Encroachments

<table>
<thead>
<tr>
<th>2018 Road KP</th>
<th>Stream Name</th>
<th>Q100 Peak Flow</th>
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<tr>
<td></td>
<td></td>
<td>Upstream Water Elev (m)</td>
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<td>Polje Creek</td>
<td>709.78</td>
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<td>87</td>
<td>Tetcela Tributary</td>
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<td>89.5</td>
<td>Tetcela Main Channel</td>
<td>270.83</td>
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Note: Velocities through bridge span vary due to differences in upstream and downstream channel geometry.
FIGURES

Figure 1  KP 6.0, Casket Creek, Model Plan View.
Figure 2  KP 6.0, Casket Creek, Existing Condition Model Results.
Figure 3  KP 6.0, Casket Creek, Alluvial Fan Split Flow Paths
Figure 4  KP 13.4, Funeral Creek, Model Plan View.
Figure 5  KP 13.4, Funeral Creek, Existing Condition Model Results.
Figure 6  KP 20.3 Sundog Tributary, Model Plan View.
Figure 7  KP 20.3 Sundog Tributary, Existing Condition Model Results.
Figure 8  KP 23.3 Sundog Creek, Model Plan View.
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Figure 10 KP 25.4 Sundog Tributary, Model Plan View.
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Figure 26 KP 121 Grainger River, Model Plan View.
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Figure 2. KP 6.0, Casket Creek, Existing Condition Model Results. Figure shows water levels near crossing centreline for 2-year and 100-year flows.
Figure 3. KP 6.0, Casket Creek, Alluvial Fan Split Flow Paths.
Figure 4. KP 13.4, Funeral Creek, Model Plan View. HEC-RAS model cross sections and 2017 site survey topography shown on 2012 orthophoto image. Flow is right to left.

Figure 5. KP 13.4, Funeral Creek, Existing Condition Model Results. Figure shows water levels near crossing centreline for 2-year and 100-year flows.
Figure 6. KP 20.3 Sundog Tributary, Model Plan View. HEC-RAS model cross sections and 2017 site survey topography shown on 2012 orthophoto image. Flow is left to right.

Figure 7. KP 20.3 Sundog Tributary, Existing Condition Model Results. Figure shows water levels near crossing centreline for 2-year and 100-year flows.
Figure 8. KP 23.3 Sundog Creek, Model Plan View. HEC-RAS model cross sections and 2017 site survey topography shown on 2012 orthophoto image. Flow is left to right.

Figure 9. KP 23.3 Sundog Creek, Existing Condition Model Results. Figure shows water levels near crossing centreline for 2-year and 100-year flows.
Figure 10. KP 25.4 Sundog Tributary, Model Plan View. HEC-RAS model cross sections and 2017 site survey topography shown on 2012 orthophoto image. Flow is bottom to top.

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Figure 12. KP 42.9 Sundog Tributary, Model Plan View. HEC-RAS model cross sections and 2018 site survey topography shown on 2012 orthophoto image. Flow is bottom to top.

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Figure 16. KP 53.2 Polje Creek, Proposed Condition Model Bridge Section. Proposed secondary crossing in left floodplain at KP 53.1 is not modelled; the simulated blockage will result in conservative results.

Figure 17. KP 53.2 Polje Creek, Proposed Condition Model 100-yr Model Results. Model output showing results for the cross section immediately upstream of the proposed bridge.
Figure 18. KP 87 Tetcela Tributary, Model Plan View. HEC-RAS model cross sections and 2018 site survey topography shown on 2012 orthophoto image. Flow is left to right.

Figure 19. KP 87 Tetcela Tributary, Existing Condition Model Results. Figure shows water levels near crossing centreline for 2-year and 100-year flows. Figure is plotted viewing upstream (opposite of normal convention).
Figure 20. KP 87 Tetcela Tributary, Proposed Condition Model Bridge Section. Figure is plotted viewing upstream (opposite of normal convention).

Figure 21. KP 87 Tetcela Tributary, Proposed Condition Model Results. Model output showing results for the cross section immediately upstream of the proposed bridge. Figure is plotted viewing upstream (opposite of normal convention).
Figure 22. KP 89.5 Tetcela Mainstem, Model Plan View. HEC-RAS model cross sections and 2018 site survey topography shown on 2012 orthophoto image. Flow is bottom to top.

Figure 23. KP 89.5 Tetcela Mainstem, Existing Condition Model Results. Figure shows water levels near crossing centreline for 2-year and 100-year flows.
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Figure 25. KP 89.5 Tetcela Mainstem, Proposed Condition Model Results. Model output showing results for the cross section immediately upstream of the proposed bridge.
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Figure 27. KP 121 Grainger River, Existing Condition Model Results. Figure shows water levels near crossing centreline for 2-year and 100-year flows.
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Photo 4  KP 13.3 Funeral Creek. Allnorth photo viewing upstream from near original crossing.
Photo 5  KP 20.3 Sundog Tributary. Allnorth photo viewing downstream from winter road.
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Photo 11 KP 42.9 Sundog Tributary. View upstream from crossing centreline, 2018.
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Photo 2: KP 6.0, Casket Creek. Ground image viewing upstream from bridge, 2018.
Photo 3: KP 13.3 Funeral Creek. View upstream to incised channel at original hairpin crossing. New crossing will be about 50 m upstream from original crossing.

Photo 4: KP 13.3 Funeral Creek. Allnorth photo viewing upstream from near original crossing.
**Photo 5:** KP 20.3 Sundog Tributary. Allnorth photo viewing downstream from winter road.

**Photo 6:** KP 20.3 Sundog Tributary. Allnorth photo viewing downstream from winter road.
Photo 7: KP 23.3 Sundog Creek. Viewing upstream at bedrock confined channel.

Photo 8: KP 23.3 Sundog Creek, Aerial Photo. Viewing upstream at bedrock confined channel.
Photo 19:  KP 25.4 Sundog Tributary, 2017. View west along road alignment, flow is left to right

Photo 10:  KP 25.4 Sundog Tributary. Incised channel with bedrock control.
Photo 11: KP 42.9 Sundog Tributary. View upstream from crossing centreline, 2018. Note very thick floodplain vegetation, both banks.

Photo 12: KP 42.9 Sundog Tributary, 2018. View downstream from crossing centreline at large left bank gravel bar. This overflow feature may be associated with main channel right bank tilted and fallen trees suggesting possible channel obstruction.

Photo 14: KP 53.2 Polje Creek. View upstream from crossing centreline.
**Photo 15:** KP 87 Tetcela Tributary. View downstream from upstream of crossing.

**Photo 16:** KP 87 Tetcela Tributary. View upstream from downstream of crossing.
Photo 17: KP 89.5 Tetcela Mainstem aerial view.

Photo 18: KP 89.5 Tetcela Mainstem view downstream.
**Photo 19:**  KP 121.0 Grainger River. Downstream from crossing, viewing upstream, 2018.

**Photo 20:**  KP 121.0 Grainger River. Viewing downstream at 2014 alignment, Hatfield KP 125.1, September 2014.
1.0 INTRODUCTION AND SUMMARY OF RESULTS

Tetra Tech Canada Inc. (Tetra Tech) was retained by NorZinc Ltd./Canadian Zinc Corporation to provide hydrotechnical design information for development of a barge crossing over the Liard River along the alignment of the proposed Prairie Creek Mine All Season Road. This work was supplemental to bridge crossing assessments which are described in a separate concurrent report.

Recommended design values are summarized below; elevations are matched to the geodetic datum values presented in Allnorth site plans for the barge site north and south landings.

- **100-year open water flood event**: discharge of 19,600 m³/s; water level 181.50 m and mean velocity of 2.4 m/s. Flow velocities along the north bank where the channel is deeper will be higher than along the south bank.

- **2-year open water flood event**: discharge of 11,000 m³/s, water level 178.6 m and mean velocity of 1.8 m/s.

- **Ice effects during breakup** are possible that will result in higher water levels. We recommend that the barge crossing landings be designed to withstand potential ice-related erosion and ice jam water levels about 1.5 m higher than the design open water level and that the ice surface will rise to some height above the water level.

- **Minimum design water level**: 172.4 m based on a late season discharge immediately prior to freeze-up.

2.0 METHODOLOGY

The assessment was conducted by transposition of Water Survey of Canada (WSC) rating curves and flow measurement information for Liard River Gauges in the vicinity of the barge crossing site.

Rating curves, daily water level summaries, and streamflow measurement summaries were obtained for three stations as follows, listed upstream to downstream.

- **WSC 10ED001**, Liard River at Fort Liard, basin area 222,000 km², continuous record 1966 to date;

- **WSC 10ED008**, Liard River at Lindberg Landing (discontinued), basin area 268,900 km², continuous record 1992 to 1996;

- **WSC 10ED002**, Liard River near the Mouth, basin area 275,000 km² continuous record 1973 to date.
The barge crossing site is located 34 km upstream of discontinued gauge 10ED008 as shown in Figure 1. This drainage area to this gauge is approximately 265,700 km² which is 1.2 times that of the active upstream gauge at Fort Liard and 0.97 times that of the active downstream gauge near the mouth.

Figure 1. Barge Crossing Location relative to WSC Gauge 10ED008.

### 3.0 SITE-SPECIFIC INFORMATION

#### 3.1 Site Survey and Bathymetry Data

Survey information for shoreline areas of the barge crossing north and south landings was obtained from plans developed by Allnorth Consultants Ltd. (Allnorth) based on a site survey completed on August 10 and 11, 2018. The water edge at the north landing is drawn at approximately road station 156+153 at elevation 173.6 m. The water edge at the south landing is drawn at approximately road station 156+668 at elevation 173.5 m.

Bathymetry for Liard River at the crossing location vicinity was surveyed on September 25, 2014, by Hatfield Consultants. Results were presented as 2.0 m interval contour lines representing water depth without a geodetic elevation reference. Figure 2 shows the bathymetry depth contours and road alignment superimposed on a Google Earth image dated August 16, 2007. Bank survey data presented in the Hatfield report identifies high water marks on the north (left) and south (right) banks that are respectively 5.51 m and 5.05 m higher than the corresponding water level(s) on the date of survey.
3.2 Liard River Discharge at Crossing on Imagery and Survey Dates.

WSC online archived data for August 16, 2007, (date of Figure 2 image) indicate Liard River mean daily flows at Fort Liard and near the mouth of 3,810 m³/s and 5,250 m³/s respectively. Applying the basin area multipliers presented above, the discharge at the barge site is expected to be in the range of 4,570 m³/s to 5,090 m³/s. A discharge of 4,900 m³/s is assumed.

WSC online archived data for September 25, 2014, (date of bathymetric survey) indicate Liard River mean daily flows at Fort Liard and near the mouth of 1,480 m³/s and 1,860 m³/s respectively. Basin area multipliers yield discharge at the barge site in the range of 1,776 m³/s to 1,804 m³/s. A discharge of 1,800 m³/s is assumed.

WSC online real-time preliminary data for August 10, 2018, (date of landing site surveys) indicate Liard River flows at Fort Liard and near the mouth of about 1,500 m³/s and 2,330 m³/s respectively. Basin area multipliers yield discharges at the barge site in the range of 1,800 m³/s to 2,260 m³/s. A discharge of 2,100 m³/s is assumed.

4.0 OPEN WATER RATING CURVES

4.1 Water Survey of Canada Data

WSC rating curve data used by WSC to convert the recorded gauge heights (stage) to an open water discharge are plotted together in Figure 3. The source data obtained from WSC have gauge heights referenced to zero depth corresponding to zero flow near the channel bottom. The curves plotted in Figure 3 for Stations 10ED001 (at Fort Liard) and 10ED002 (near the mouth) use the gauge height values reported by WSC, while the curve for Station 10ED008 (Lindberg Landing just below the barge crossing) is lowered by 4.2 m. The offset value for the Lindberg Landing curve was selected so that the curve plots more closely to those from the other stations and can be extrapolated to higher discharges using the relationships from those other curves.
While the three stage-discharge curves shown in Figure 3 are similar, the velocity-discharge data did not yield the same consistency. For a given discharge, the mean flow velocity for Station 10ED001 at Fort Liard is about 50% greater than at Lindberg Landing. This difference is most likely related due to a channel section at Lindberg Landing that is deeper than at Fort Liard. A significant difference in channel depth would be consistent with the gauge height offset required to bring the stage-discharge rating for 10ED008 closer to that for the other two stations.

Channel flow area data for the barge crossing were estimated from the bathymetry shown in Figure 2, surveyed on Sept 25, 2014 at a discharge of about 1,800 m$^3$/s. The open water top width, measured perpendicular to the river, is about 450 m. The maximum water depth is about 12 m and the average depth is about 6 m. This information yields a flow area of about 2,760 m$^2$ and mean velocity of 0.67 m/s. This velocity is reasonably consistent with the velocity-discharge relationship shown in Figure 4 for Lindberg Landing (computed curve has a velocity of 0.74 m/s for a discharge of 1,800 m$^3$/s). The close similarity to this gauge, and not the upstream gauge at Fort Liard, is probably associated with the geomorphic effects of the South Nahanni River which joins the Liard just upstream of the barge crossing. Figure 5 shows the confluence.

It is noted that the bathymetry approaching the barge crossing site has relatively deep water along the north bank, probably because of the upstream confluence. The flow velocity at the crossing is also expected to be asymmetrical, with higher velocities in the north portion of the channel.
Figure 4. Liard River Velocity-Discharge Relationships

\[ y = 0.0192x^{0.4876} \]

Figure 5. Liard River at South Nahanni River Confluence
4.2 Breakup Period and Annual Minimum Water Levels

In northern climates, maximum water levels can occur during breakup of the river ice. Breakup levels can be significantly higher those during peak discharges which typically occur during open water conditions. Minimum discharges typically occur during winter ice cover conditions, when the rating curves developed for open water conditions do not apply; the minimum water levels therefore need to be assessed by direct evaluation of the water level data.

Tetra Tech’s assessment of breakup period water levels on the Liard involved review of the daily water level records recorded by WSC for the three Liard River gauges identified above.

Our summary of finding is:

1. For downstream Station 10ED002, Liard River near the mouth, annual maximum water levels are often greatest for breakup conditions. However, these are understood to be influenced by backwater from Mackenzie River immediately downstream and therefore do not reflect conditions at the upstream barge crossing location.

2. For Station 10ED008, Liard River at Lindsberg Landing, there are no instances of breakup water levels that are greater than same-year water levels for ice-free conditions. However, because there are only five years of annual level data, this result is not a reliable predictor of the potential for infrequent ice jams that could result in elevated water levels.

3. For Station 10ED001, Liard River at Fort Liard, a reasonably complete record of daily water levels is available for years 1981 to 2016. Within this period, the maximum open water discharge was 15,300 m$^3$/s on June 11, 2012, with a corresponding stage of 9.983 m. During the same period there have been occurrences of higher stages, presumably associated with ice conditions. The highest stages listed in descending order are: 11.461 m on May 5, 2010, 10.779 m on Apr 29, 2006, 10.727 m on May 2, 1992, and 10.572 m on May 5, 2009.

While the frequency of ice jam events at the barge crossing is not known, we recommend that the barge crossing landings be designed to withstand potential ice-related erosion and ice jam water levels about 1.5 m higher than the design open water level. The design should also anticipate that the ice surface will rise to some height above the water level.

Minimum daily water levels were assessed using online data available for years 2002 to 2016. For Station 10ED001 (Fort Liard), the minimum levels typically occur in late October or early November, prior to the development of an full ice cover. The lowest level was 2.235 m reported on October 25, 2012, with a corresponding flow of 815 m$^3$/s. The same-day discharge for Station 10ED002 (near the mouth) was 1,400 m$^3$/s. The date corresponds to the onset of ice backwater conditions at both gauges when the open water rating still has some validity.
5.0 BARGE SITE CHARACTERISTICS

5.1 Open Water Rating Curve

Figure 6 shows the recommended stage-discharge rating curve for the barge site, referenced to geodetic water elevation, that is applicable to the north and south landing locations identified on site drawings prepared by Allnorth in 2018. This curve follows the rating relationship shown in Figure 3 for Station 10ED008 and is extended for higher discharges by following the relationship for Station 10ED002. The offset between the gauge height values shown in Figure 3 and the water level elevations shown in Figure 6 is based on a single match point defined by the conditions as of the August 10, 2018 survey date: a water level elevation of 173.5 m for the assumed discharge of 2,100 m$^3$/s described above. The offset from the gauge height amounts in Figure 3 to the water level elevation at the barge crossing is 170.2 (173.5 minus 3.3).

The curve shown is based on the WSC information and none of the standard curve fit equations (log, power, linear, etc.) gave a good fit to the data points. A single equation to describe the relationship is therefore not provided.

![Open Water Rating Curve for Liard River at CZN Barge Crossing](image)

5.2 Design Flows, Levels, and Velocities

Design open water flows for the barge crossing have considered the statistical 100-year discharges for the Liard River stations at Fort Liard and at the Mouth, and also the station record lengths. For both stations, the highest flood of record occurred in 2012.

- The 2012 reported peak discharge at Fort Liard, 15,300 m$^3$/s is the highest in 67 years of record and has a preliminary return period just over 1 in 100-years. The statistical 100-year peak discharge for Fort Liard is 15,400 m$^3$/s.
The 2012 reported peak discharge near the mouth, 19,600 m³/s, is the highest in 45 years of record and has a preliminary return period of about 1 in 200 years. The statistical 100-year peak discharge is 18,600 based on a Log Pearson 3 distribution.

We have relatively high confidence in the reliability of the 2012 return period estimate as being a 100-year flow based on the Fort Liard data because the record is longer than for the station near the mouth. 2012 is also the flood of record, and is also therefore a good candidate for a design flood regardless of the computed return period at the mouth.

If the 2012 peak flows at the barge crossing (and Lindberg Landing) is estimated from the Fort Liard recorded data adjusted for basin area (1.2 multiplier) the result is 18,360 m³/s. The estimate made from the measured flow at the mouth, adjusted for basin area (0.95 multiplier) is 18,600 m³/s. However, our recommended design flow also considers the probable major flow contribution from the South Nahanni River and the uncertain reliability of the area multipliers for peak flow events.

Peak flows at the barge crossing (and Lindberg Landing) may be very similar to those at the mouth because the barge site is just below the South Nahanni which is the last of the large high-runoff mountain basins. A check on reported peak flows for Lindberg Landing (data 1992-1996) versus flows near the mouth found very little difference in peak flows for larger events. The largest reported peak flow for 1992 (12,800 m³/s at Lindberg Landing) was within 1% of the downstream peak (12,700) near the mouth. The discharges for the next highest year were the same at both stations.

We recommend adopting the 19,600 m³/s flood of record near the mouth (100-year flood event at upstream Fort Liard) as the design event directly applicable to the barge crossing. Based on the rating relationships shown in Figures 4 and 6, this would have a water elevation of 181.5.0 m and a mean velocity of about 2.4 m/s. Flow velocities along the north bank where the channel is deeper will be higher than along the south bank.

Using a similar approach, the Liard River 2-year flood (50% probability of exceedance) at the mouth, and applied directly to the barge site, would have a discharge of 11,000 m³/s, water level of about 178.6 m and mean velocity of about 1.8 m/s.

As discussed above, higher water levels are possible due to ice effects during breakup. We recommend that the barge crossing landings be designed to withstand potential ice jam water levels about 1.5 m higher than the design open water level and that the ice surface will rise to some height above the water level.

A minimum design water level would be about 172.4 m based on a late season open water discharge of about 1,200 m³/s immediately prior to freeze-up. This discharge is slightly less than the minimum reported open water flow at the Lindberg Landing gauge of 1,400 m³/s over five years of record.

### 6.0 LIMITATIONS

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7.0 CLOSURE

We trust this report meets your present requirements. If you have any questions, please contact the undersigned.

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