Clinton Creek Remediation Project

Dam Breach Assessment
Project #VE52705E

Prepared for:

Government of Yukon Energy, Mines and Resources
Assessment and Abandoned Mines
2C – 4114 4th Avenue, Whitehorse, Yukon, Y1A 4N7

15 November 2019

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2C – 4114 4th Avenue, Whitehorse, Yukon, Y1A 4N7

Prepared by:

Wood Environment & Infrastructure Solutions
#600 – 4445 Lougheed Highway
Burnaby, BC V5C 0E4
Canada
T: 604-294-3811

15 November 2019

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List of Acronyms and Abbreviations

AAM	Assessment and Abandoned Mines
AEP	Annual exceedance probability
CIRNAC	Crown-Indigenous Relations and Northern Affairs Canada
CCRP	Clinton Creek Remediation Project
DS	Drop structure
IDF	Inflow design flood
IPRP	Independent Project Review Panel
PMF	Probable Maximum Flood
TA	Task Authorization
TH	Tr'ondëk Hwëch'in
Wood	Wood Environment & Infrastructure Solutions, a Division of Wood Canada
YG	Yukon Government

1.0 Introduction

1.1 Wood Project Scope

Wood Environment & Infrastructure Solutions, a Division of Wood Canada Ltd. (Wood) was retained by the Yukon Government (YG) to provide engineering services to inform a 10% design and cost estimate for six options (three each for Clinton Creek and Wolverine Creek) under the Clinton Creek Remediation Project (CCRP). Wood's work has been authorized by YG under master services agreement (MSA) #C00034680.

This project is being managed by YG with project partners Crown-Indigenous Relations and Northern Affairs Canada (CIRNAC) and the Tr'ondëk Hwëch'in (TH) First Nation. Additional technical input and recommendations are provided to the project partners though an Independent Project Review Panel (IPRP).

1.2 Option Descriptions

The Project Partners' descriptions of the six candidate options specified in Yukon's original scope document (Yukon 2017) were as follows. Note that references to LCCA (Life Cycle Cost Analysis) options in these descriptions are taken from option definitions applied in a 2014 estimating exercise completed for Yukon in 2014 (Worley Parsons 2014).

Clinton Creek Side Closure Concepts:

a) **Water Passage and Catastrophic Failure Mitigation (LCCA Options D3, I2) (CC1 in Wood reports)** - Conduct sufficient work on the waste rock pile to mitigate a catastrophic failure of the pile and construct a water conveyance channel to provide water passage from Hudgeon Lake to Clinton Creek.

b) **Water Passage, Catastrophic Failure Mitigation and Lowering Lake (LCCA Option E3) (CC2 in Wood reports)** - Conduct sufficient work on the waste rock pile to mitigate a catastrophic failure, construct a water conveyance channel to provide water passage from Hudgeon Lake to Clinton Creek and lower Hudgeon Lake as part of that concept.

c) **Water Passage with a Reduction of the Lake Level, Dam Elimination, and Catastrophic Failure Mitigation (LCCA Option F) (CC3 in Wood reports)** - Conduct sufficient work on the waste rock pile to prevent it from acting as a dam (i.e., as defined by the Canadian Dam Association) on Clinton Creek and to mitigate a catastrophic failure of the waste rock pile. Construct a water conveyance channel to facilitate water passage through the site.

Wolverine Creek Side Closure Concepts:

a) **Sediment Control Only (Not in the LCCA) (WC1 in Wood reports)** - Construct a sediment control structure downstream from the rock-lined channel in Wolverine Creek. No work on the tailings pile or channel is required.

b) **Water Passage and Stability Improvements (LCCA Options B, C, D, D2 - note that Option B does not apply a remediation measure for the tailings) (WC2 in Wood reports)** - Conduct sufficient work at the base of the tailings pile to minimize tailings movement and to form a semi-stable surface to construct a water conveyance channel.

c) **Asbestos Isolation (LCCA Options E, E2) (WC3A&B in Wood reports)** - Stabilize the tailings pile to allow a cover to be placed or relocate the tailings pile.

1.3 Breach Assessment Report Scope

This report has been prepared to describe the breach assessment undertaken to inform the above noted 10% design phase at Clinton Creek. As such, the study does not meet all of the requirements that may be expected for a full dam breach assessment of a constructed dam. The assessment was intended to provide sufficient information for decisions to be made as follows:

1. For CC1, as described in a) Section 1.2, a critical facet of the spillway design was to establish reasonable and compliant design criteria for the spillway. Canadian Dam Association Guidelines (CDA 2014) **Table 4-1** sets specific target criteria for Closure – Passive Care depending on the flood hazard involved.
2. To inform the design of CC2, as described in b) Section 1.2, with regards to the design criteria as stated above and to provide an indication of the consequences of a breach of the channel as it is currently designed.

The background information that is provided in Appendix A provides additional context for the development and evolution of the breach assessment scope.

Wood acknowledges that the scheme designed for WC2 includes the creation of a new buttress dam and the subsequent impoundment of water behind it on Wolverine Creek. The assessment did not involve a review of consequences of the failure of this option. The requirement was for a "semi-stable" surface to construct a water conveyance channel. The scale of the required spillway and variations resulting from the designed flood selection are minor relative to the project's overall complexity. If this option is selected, a full and detailed breach assessment would be required during the preliminary design phase. Furthermore, the concept of a "semi-stable" surface for water conveyance is incongruous with the requirements of CDA dam safety guidelines, and as such, these matters would need to be resolved in later design phases.

1.4 Comments Log

The Comments Log that is included with this document describes the disposition of Partner comments on the draft submission of this report. The context provided by these responses provides support to the review and interpretation of this document's content.

2.0 Site Description

2.1 Project Location

The Clinton Creek Mine Site (the Site) is a former asbestos mine that was operated between 1968 and 1978. The site is located approximately 100 km northwest of Dawson City, Yukon near the confluence of the Fortymile and Yukon Rivers (**Figure 2-1**). The site is accessed from the Top of the World Highway (Yukon Highway 9) and then Clinton Creek Road. These roads are typically maintained between the months of June and September when the George Black River Ferry runs between East and West Dawson. During the fall and winter months, the site is only accessible by helicopter or snowmobile.

wood. Environment & Infrastructure Solutions a Division of Wood Canada Limited #600 - 4445 Lougheed Highway Burnaby, BC V5C 0E4 Tel. 604-294-3811 Fax 604-294-4664	DWN BY: KL CHK'D BY: GG DATUM: NAD 83 PROJECTION: UTM Zone 7 SCALE: AS SHOWN	CLIENT / PROJECT: 2019 10% DESIGN PHASE TITLE: SITE LOCATION PLAN	DATE: NOV 2019 PROJECT NO: VE52705E REV. NO: A FIGURE NO: 2.1

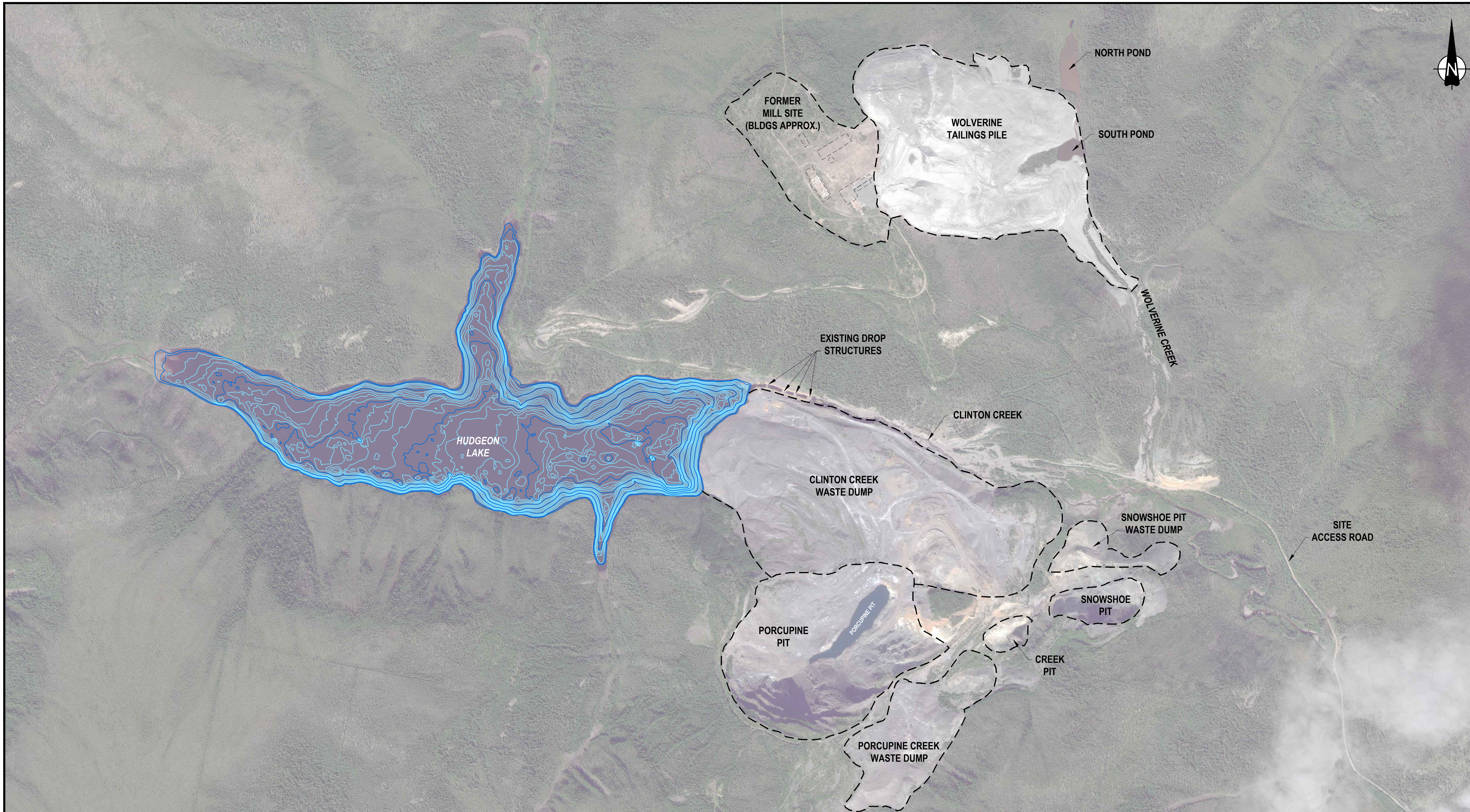
2.2 Site Description

Major elements of the site are shown in **Figure 2-2**. During mine operations, material was removed from three ore sources: the Porcupine Pit (the largest pit), Horseshoe Pit and the Creek Pit. Waste was placed in the following locations:

1. The Clinton Creek Waste Dump, where waste was placed along the south valley wall of the Clinton Creek valley. It is estimated that 60 million tonnes of waste were placed in the Clinton Creek Waste Dump;
2. The Porcupine Creek Waste Dump, where waste was placed into the Porcupine Creek valley (Porcupine Creek Waste Dump); and
3. The Snowshoe Pit Waste Dump, where waste was placed on the north side of the Snowshoe Pit along the upper edge of the south Clinton Creek valley wall.

During mining operations, ore was transported from the south side of the Clinton Creek valley to the mill site located on high ground on the north side of Clinton Creek and along the upper edge of the west valley wall of Wolverine Creek via an aerial tramway. The ore, a serpentine rock containing chrysotile asbestos, was processed in the mill while the waste material or tailings were transported via conveyor to two piles positioned along the steep western slope of Wolverine Creek with one pile located north of the other. Approximately 12 million tonnes of tailings were deposited into these two piles.

In 1974, waste material deposited onto the south slope of the Clinton Creek valley, forming the Clinton Creek Waste Dump, is believed to have failed, blocking the Clinton Creek flow path. It should be noted that Clinton Creek was diverted north of the creek's natural path, which flowed along the toe of the south slope of the Clinton Creek valley prior to the failure of the dumpsite. The failure created a landslide dam, which impounded water upstream, producing what is now known as Hudgeon Lake. Additional information about the formation of Hudgeon Lake is provided in Amec Foster Wheeler (2018a). It is currently believed that only a portion of the Clinton Creek Waste Dump failed and that efforts were made to stabilize the resulting landslide dam. Currently, water discharging from Hudgeon Lake travels southeast approximately 8 km downstream via Clinton Creek to Fortymile River. The outlet from Hudgeon Lake was, through four gabion drop structures (DS1, DS2, DS3 and DS4), constructed between 2002 and 2004. DS4 was upgraded and repaired in 2015 following damage sustained in 2010. Damage to DS4 was noted in the field following the 2018 spring freshet, and additional damage was caused to the drop structure during a flood event in August 2018.



NOTE:
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wood.

Environment & Infrastructure Solutions
a Division of Wood Canada Limited
#600 - 4445 Lougheed Highway
Burnaby, BC V5C 0E4
Tel. 604-294-3811 Fax 604-294-4664

CLIENT:

Yukon

DWN BY: TH	PROJECT: CLINTON CREEK MINE		DATE: JUNE 2019
CHK'D BY: KH			PROJECT NO: VE52705E
DATUM: NAD 83	TITLE: SITE LAYOUT		REV. NO: A
PROJECTION: UTM Zone 7			FIGURE NO: 2.2
SCALE: AS SHOWN			

P:\CAD\CIV\2018 Site Invetigation PDF\Figure-1.2.dwg - FIG 1.2 - Jul. 02, 2019 11:42am - Juliana.Matos

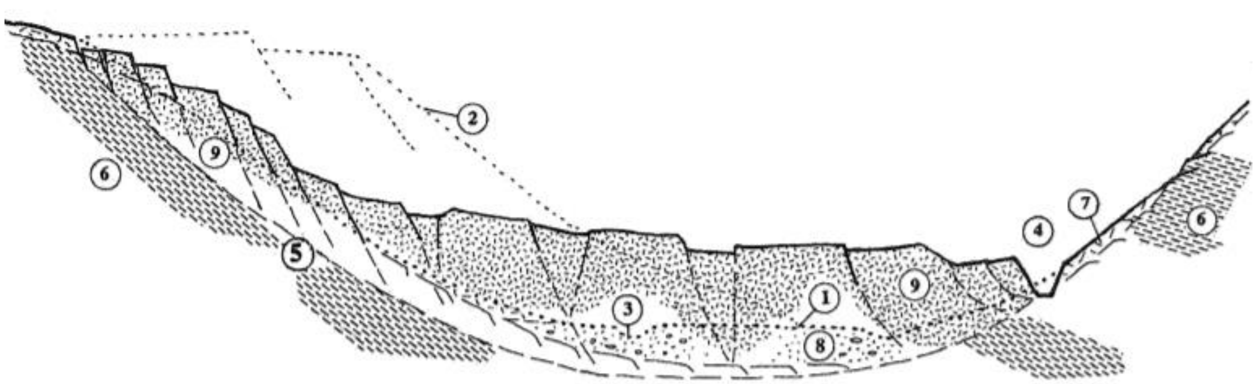
2.2.2 Geology

A detailed description of the geological setting is provided in the *Geological & Geotechnical Site Characterization and Model* report (Wood 2019). As the site characterisation report forms part of the deliverables for this overall work package, the reader is referred to this document for detailed descriptions of the surficial geology.

A good summary of the local geology is also provided by Stepanek and McAlpine (1992), who describe the bedrock as largely covered with overburden. The soil cover is composed of colluvium on the slopes and alluvium in the valley bottoms. The valley bottoms form a very weak foundation for the waste dumps, being primarily composed of alluvial deposits with organics and ice-rich silts (Stepanek and McAlpine 1992).

2.2.3 Description of the Dam

A review of historical air photos shows that the Clinton Creek valley was originally broad (approximately 250 m) and flat with a meandering stream in the valley bottom similar in form to the current stream immediately downstream from the waste rock dump near the confluence with Wolverine Creek. Stepanek and McAlpine report that the steep hillside and weak foundation precipitated a deep foundation failure under the waste rock dump and gradual sliding across the valley bottom and up the north valley wall. The resultant dam is approximately 250 m wide and 800 m long.



1-Original ground surface, 2–Initial configuration of the waste dump, 3–Original creek channel, 4–Creek channel (1992), 5–Probable slip plane, 6-Bedrock (shale), 7-Colluvium, 8-Alluvium, 9-Slide debris

Figure 2-3: Landslide Dam Cross-Section (Source: Stepanek and McAlpine 1992)

Figure 2-4 below presents an aerial photograph of the dam taken during a site visit completed by Wood in June 2019.

Figure 2-4: Clinton Creek Waste Rock Dump Dam (Source: Wood 2019)

2.2.4 Description of Waste Rock

"The waste rock consists mainly of argillite, phyllite, platy limestone and micaceous quartzite. Shale is commonly disintegrated into silt sized and platy sand or gravel sized particles which form the matrix of the waste material" (excerpt from Stepanek and McAlpine 1992).

Wood conducted sieve tests during drilling programs in 2016 and 2018, and the results are shown in **Figure 2-5** below. The results were used to inform decision making on headcut erosion rates and depths and for sediment transport modelling.

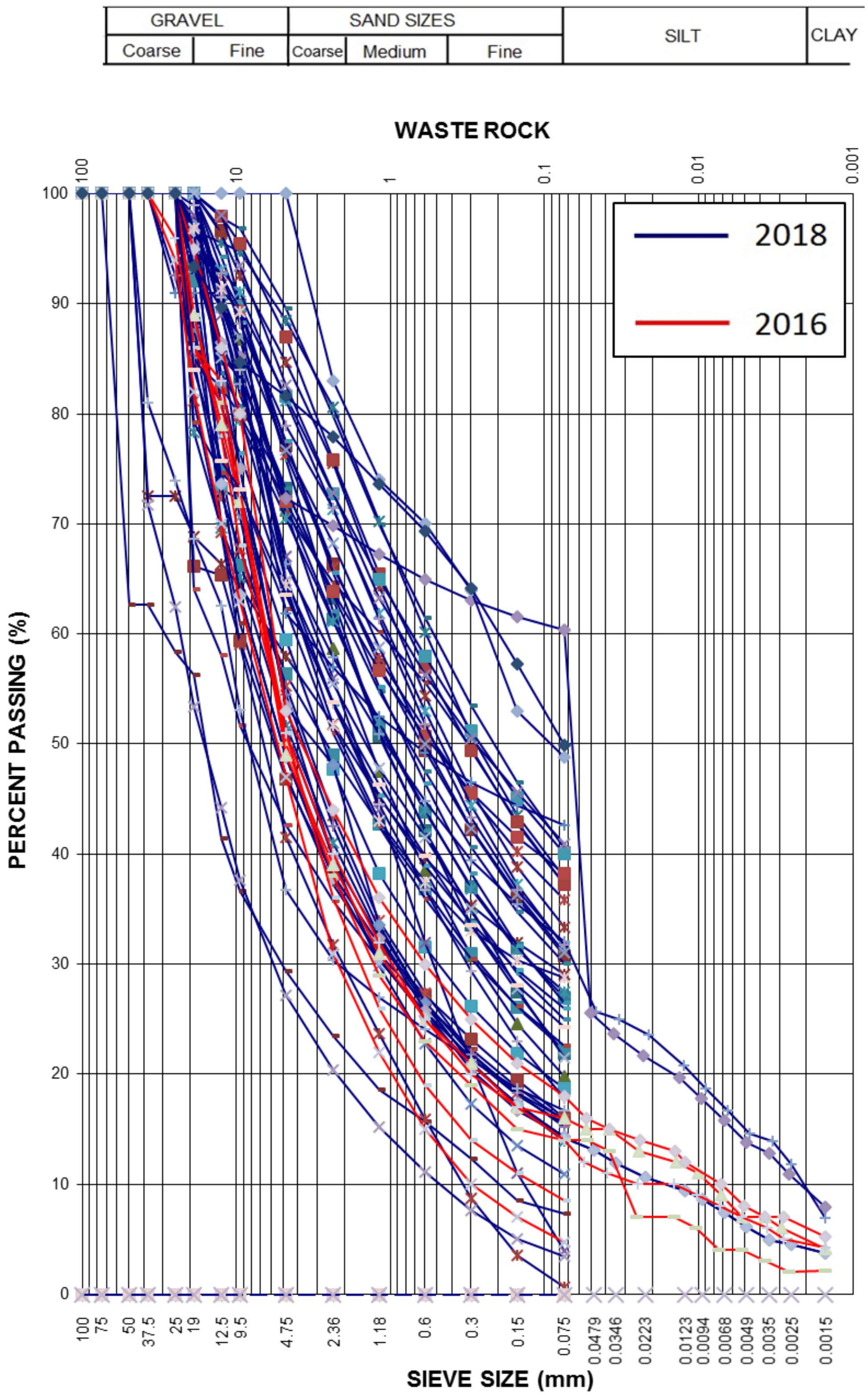


Figure 2-5: Clinton Creek Waste Rock Gradation (Source: Wood 2019)

3.0 Literature Review for Landslide Dam Break Flood Estimation

The estimation and evaluation of floods resulting from a potential dam breach necessitated a review of literature related to the formation and failure of landslide dams. Research paper results on the failure of constructed earthen dams were also used to inform the estimates. A list of documents reviewed is included in the references provided at the end of this report. Evans (1986) provided insight into discharge produced through outburst flooding resulting from the breaching of man-made and natural dams and compared his results with Clague and Matthews' (1973) findings on jokulhlaups[1], identifying a relationship between outburst volumes and maximum discharge during a breach. Evans found natural debris dams formed by landslides as well as man-made rock and earthfill dams to breach in a similar manner. The maximum discharge generated from an outburst flood (Q_{max}) from a man-made reservoir is related to the volume discharged (V_{max}) during the event according to the regression equation (r^2=0.836):

$$Q_{max} = 0.72V_{max}^{0.53}$$

Costa and Schuster (1987) provide a regression equation for estimating the peak discharge for the failure of earth/rockfill, landslide, moraine and glacier dams. The equations are based on the potential energy (PE) of the reservoir, which is the product of the height of the dam, volume of water and specific weight of water (9800 newtons/m^3). The equation relevant to this investigation pertains to landslide dams and is given as:

$$Q_{max} = 0.0158PE^{0.41}$$

Other empirical equations developed for estimating flow generated from breached embankment dams were considered, including Gupta and Singh's (2012):

$$Q_p = 0.02174V^{0.4738}h^{1.1775}(W + L)^{0.17094}$$

where: V=volume released; h =height; W = width; L = average embankment length

and Rico's (1988):

$$Q_{max} = 6.3H^{1.59}$$

In a review of the formation and failure of natural dams, Clague and Evans (1994) state that most historical landslide dams in the Canadian Cordillera failed within hrs or days of forming by overtopping and subsequent incision. The escaping flood waters first erode the toe of the dam and subsequently cut headward into the lake. **Figure 3-1** below presents a comparison between failed landslide dams in the Canadian Cordillera and those worldwide (Costa and Shuster 1988). The figure shows that approximately 90% of dams that have failed did so within the first roughly 25 days with the majority failing within the first few days.

Overtopping waters erode the toe of the dam and then headcut through the dam to breach. While failure may occur quickly enough to spur significant flooding, applying the height-to-width ratio is typically sufficient in providing a measure of protection against this result (Clague and Evans 1994).

[1] A jokulhlaup is a flood resulting from the failure of a glacial impoundment.

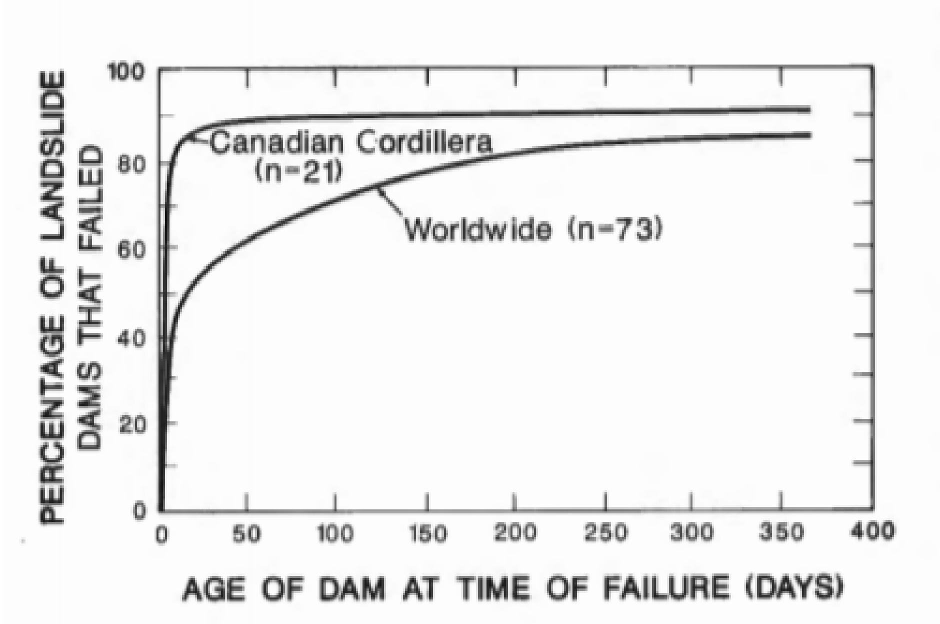


Figure 3-1: Age of Dam at Time of Failure (Source: Clague and Evans 1994)

3.1 Empirical Equation-Based Flood Estimates

3.1.1 Breach Parameters

The above referenced empirical equations require an estimate of reservoir volume and/or the maximum depth of breach. The estimation of these parameters requires considerable discretion on the engineer's part, as such parameters have a significant effect on the magnitude of an outburst flood.

An ultra-conservative approach would involve assuming the occurrence of a full-depth breach down to the original creek bed level. Based on site observations and the history of flood damage to DS4, in Wood's opinion, a potential breach scenario would either involve head-cutting back through the gabion structures, outflanking the gabions or a combination of the two causing a breach through or down the side of the drop structures. The depth of breach was estimated using two different methods.

First, it is assumed that the channel would headcut at a slope similar to that of the downstream channel, which appears to be in regime. The downstream channel is on a slope of approximately 1% to 2%. Assuming that the headcut is initiated at the toe of DS4, the resulting breach elevations were estimated as 399.68 m (for a 1% headcut) and 401.91 m (for a 2% headcut).

Second, the sediment transport capability of Clinton Creek was modelled to determine how much downcutting would occur using the existing waste rock gradation shown in **Figure 2-5** (i.e., the channel is unarmoured). This modelling is described in further detail in Section 7. Under the most extreme flood conditions tested (1/3 between a 0.1% AEP flood and PMF), downcutting at the lake outlet was recorded as 7.1 m. The second method therefore assumes a 7.1 m depth of breach.

3.1.2 Empirical Method Results

The calculated outburst flood determined from various empirical methods is provided in **Table 3-1** below.

Table 3-1: Calculated Flood Discharges

Method	Full Depth Breach (m^3/s)	1% Headcut (m^3/s)	2% Headcut (m^3/s)	7.1 m Breach Depth (m^3/s)
Costa & Schuster (1988)	**1888**	**1214**	**1051**	**813**
Gupta & Singh (2011)	6519	1919	1414	1020
Evans (1986)	3668	3033	2797	2407
Rico (1988)	1052	336	245	142

As noted in Section 3, Clague and Evans state that the most common failure mode for landslide dams is that of headcutting. Given the length and width of the landslide dam, the occurrence of a full depth breach by this mechanism seems improbable. Given observations of flood damage to the drop structures, the most likely scenario should involve headcutting at some gradient from the toe of DS4. The values calculated using Rico (1988) were judged to be insufficiently conservative. Costa and Schuster's equation specific to landslide dams was adopted in Clague and Evans (1994), and as such the values shown in bold above are deemed most representative of the empirical methods reviewed.

4.0 Dam Classification

Previous studies reported by TTEBA (2016) and Amec Foster Wheeler (2017) have presented discussion that has led to the development of a "Significant" risk classification. The CDA Guidelines recommend a classification system based on an assessment of consequences. Consequences of dam failure may include loss of life, property and environmental damage, and general disruption to the population in the inundated area. In addition, the release of stored tailings, accumulated silt or impacted waters may have detrimental environmental effects, including impacts on aquatic habitat, recreational property and activities, and various infrastructure. **Table 4-1** presents this approach.

Table 4-1: CDA Classification System

Dam Class	Population at Risk [Note 1]	Incremental Losses		
		Loss of Life [Note 2]	Environmental and Cultural Values	Infrastructure and Economics
Low	None	0	Minimal short-term loss No long-term loss	Low economic losses; area provides limited infrastructure or services
Significant	Temporary Only	Unspecified	No significant loss or deterioration of fish or wildlife habitats Loss of marginal habitats only Restoration or compensation in kind highly possible	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes

Dam Class	Population at Risk [Note 1]	Incremental Losses		
		Loss of Life [Note 2]	Environmental and Cultural Values	Infrastructure and Economics
High	Permanent	10 or fewer	Significant loss or deterioration of important fish or wildlife habitat Restoration or compensation in kind is highly possible	Significant economic losses affecting infrastructure, public transportation, and commercial facilities
Very high	Permanent	100 or fewer	Significant loss or deterioration of critical fish or wildlife habitat Restoration or compensation in kind possible but impractical	Very significant economic losses affecting important infrastructure or services (e.g., highways, industrial facilities, and storage facilities for dangerous substances)
Extreme	Permanent	More than 100	Major loss of critical fish or wildlife habitat Restoration or compensation in kind impossible	Extreme losses affecting critical infrastructure or services (e.g., hospitals, major industrial complexes, and major storage facilities for dangerous substances)

Source: Dam Classification (Table 2-1, CDA 2007)

Notes

Note 1. Definitions for population at risk:

None – No identifiable population is at risk, so there is no possibility of loss of life other than through unforeseeable misadventure.

Temporary – The population is only temporarily located in the dam-breach inundation zone (e.g., seasonal cottage use, travel on transportation routes, participation in recreational activities).

Permanent – The population at risk is ordinarily located in the dam-breach inundation zone (e.g., as permanent residents); three consequence classes (high, very high, and extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist with decision-making if the appropriate analysis is carried out).

Note 2. Implications of loss of life:

Unspecified – The appropriate level of safety required at a dam where the local population is temporarily at risk depends on the number of people present, their exposure time, the nature of their activity, and other conditions. A higher class could be appropriate depending on the relevant requirements. However, design flood standards, for example, may not be higher when the temporary population is not likely to be present during the flood season.

TetraTech EBA previously completed a preliminary dam classification for the site (Tetra Tech EBA 2016), and the dam classification for the Clinton Creek Waste Dump was deemed "Significant" based on the following assumptions:

- There is no permanent population at risk;
- The habitat at risk is marginal and could be restored; and
- Only recreational facilities, seasonal workplaces, and infrequently used transportation routes are at risk.

As part of the same preliminary dam classification, a dam classification of "Significant" was also assigned to the Wolverine Tailings Pile. Analyses leading to the consequence assessment and classification of dams generally include the characterization of a hypothetical dam breach, flood wave routing, inundation mapping, and impact evaluation. A wide range of methods could be applied in each of these phases with the approach selected depending on the information available and the level of detail expected. A dam breach and inundation study was carried out as part of a risk assessment report for both the Clinton Creek Waste Dump and the Wolverine Tailings Pile (UMA Engineering Ltd. 2000) and was updated in 2016 (Tetra Tech EBA 2016). The information was reviewed and deemed sufficient in Amec Foster Wheeler (2017) to make an informed assessment of the consequence classification for both the Clinton Creek Waste Dump and Wolverine Creek Tailings Pile. Amec Foster Wheeler (2017, Table 5.2) also provided a more detailed analysis of the factors involved and supported the conclusions reached by TetraTech EBA (2016).

5.0 Hydrology

5.1 Clinton Creek Basin Description

Clinton Creek is positioned approximately 117 km^2 from the outlet of Hudgeon Lake. The upper reaches of the basin are located in unglaciated terrain in Alaska. The headwaters originate on the flanks of the 1290-m-high Forty Mile Dome across the Alaska border. In the headwaters, the gradient of Clinton Creek is approximately 5%, easing to roughly 1.7% to 2% below 550 m in elevation. Between 550 m in elevation and Hudgeon Lake, Clinton Creek is a meandering stream that winds down the 200-m-wide valley bottom. The channel appears to be in regime upstream from Hudgeon Lake, and it appears that sufficient time has passed since the formation of the lake for the channel to find regime in the downstream environment as well.

Despite the relatively slack gradient of Clinton Creek at least in the lower reaches, the basin will respond quickly to rainfall due to the dendritic stream network, mountainous topography and relatively sparse vegetation conditions in the area.

The basin does not include an active Water Survey of Canada flow gauging station.

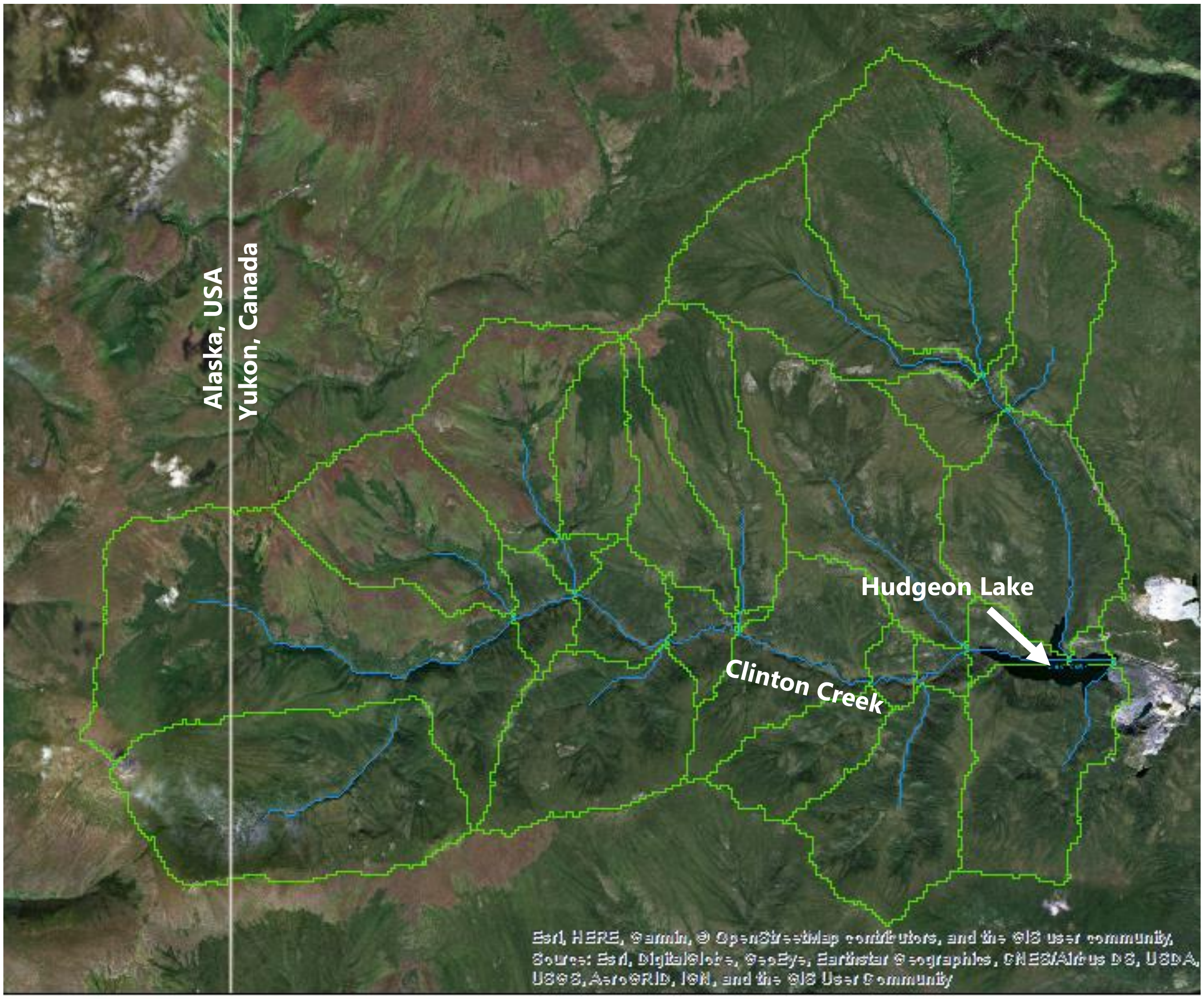


Figure 5-1: Clinton Creek Basin

5.1.1 Return Period Peak Flows

Single station flood frequency analyses of peak flows were performed using the Extreme Value distribution for nine hydrometric stations (**Table 5-1**) to obtain estimates of 4%, 2%, 1%, 0.5% and 0.1 AEP flows. These flows were converted to unit discharges and then plotted in EXCEL on log-log paper (**Figure 5-2**). A trend line was fitted to the data in the form of a power equation:

$$q_{return\ period} = C\ A^b$$

$q_{return\ period}$ is the unit low flow for a specific return period,

C is a constant,

A is the drainage basin area in km^2, and

b is an exponent related to the slope of the trend line.

Table 5-1: Stations Used to Derive Statistical Flow Estimates

Station Number	Station Name	Province / State	Latitude	Longitude	Approx. Distance from Clinton Creek Mine Site km	Year From	Year To	Drainage Area, km^2	No. of Recorded Max Inst. Peak Discharges	Used in UMA (2000) Report
09EB003	Indian River above the mouth	Yukon T	63.77	-139.63	94	1990	1991	2210	21	Yes
09EA004	North Klondike River near the mouth	Yukon T	64.00197	-138.596	116	1974	2015	1090	33	Yes
09EA003	Klondike River above Bonanza Creek	Yukon T	64.04278	-139.408	79	1990	1991	7810	32	Yes
10MA003	Blackstone River near Chapman Lake airstrip	Yukon T	64.90139	-138.276	85	1984	2014	1180	16	No
09EB004	Sixty Mile River near the mouth	Yukon T	63.68939	-140.16	90	2011	2014	3060	16	No
09EC002	Fortymile River near the mouth	Yukon T	64.39722	-140.611	8	1982	1996	16600	11	No
15344000	King Creek near Dome Creek	Alaska	64.39389	-141.412	34	1975	2010	15.5	36	Yes
15470300	Lower Jack Creek near Nabesna AK	Alaska	62.54417	-143.323	249	1975	2008	17.6	34	Yes
15305920	WF TR Near Tetlin Junction AK	Alaska	63.6675	-142.267	116	1967	1997	1.6	26	Yes

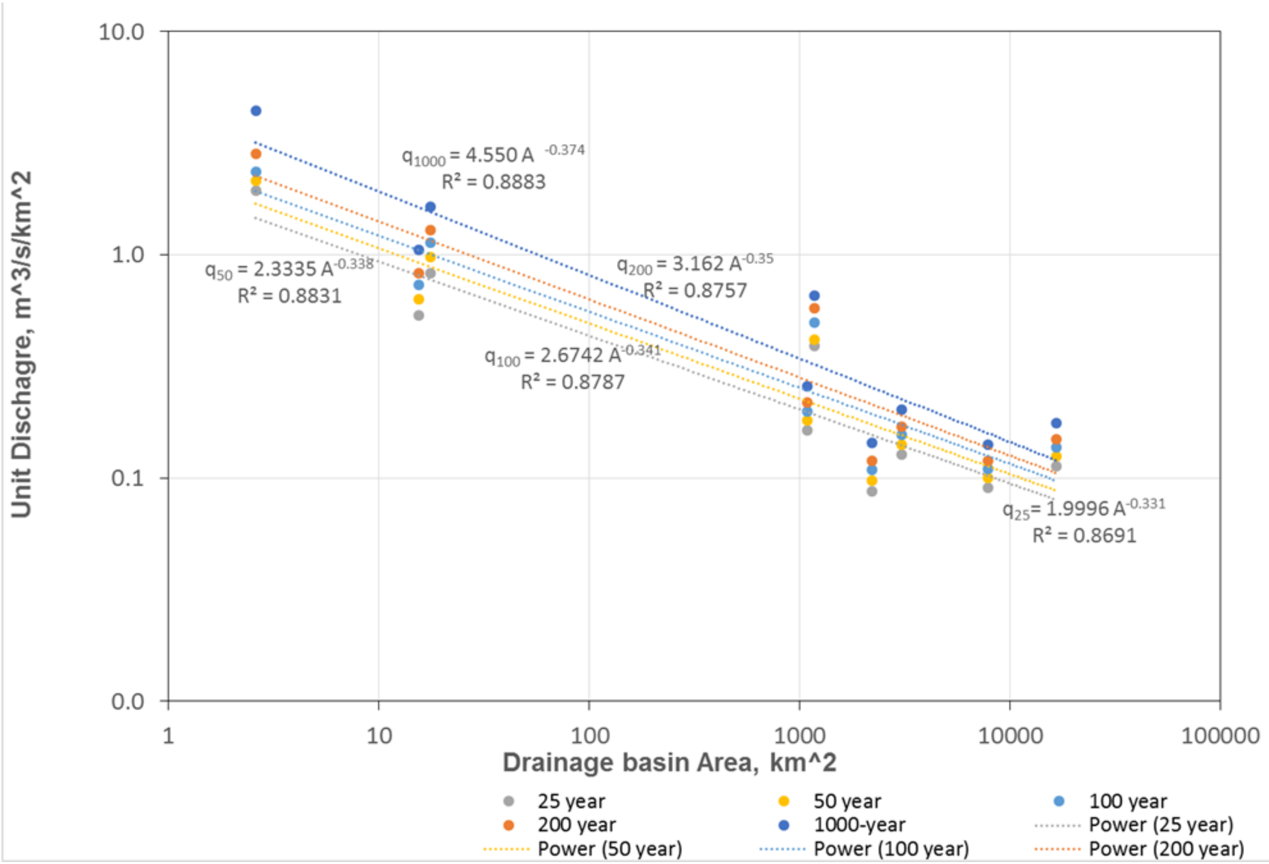


Figure 5-2 Unit Discharge Versus Drainage Basin Area

The results are presented in **Table 5-2** below.

Table 5-2: Return Period Design Flows

Flood Probability	Main Stem Clinton Creek at Hudgeon Lake Inlet	Easter Creek at Hudgeon Lake	Clinton Creek at Hudgeon Lake Outlet	Clinton Creek Upstream from Wolverine Creek	Clinton Creek Downstream from Wolverine Creek	Clinton Creek at Road Crossing	Wolverine Creek at Mouth
AEP	Drainage Basin Area in km^2 (Source: UMA Engineering 2000)						
	63.1	26.3	111.9	116.6 (117.2)*	145.2	203.8	28.6
	Flow in m^3/s						
4%	32.0	17.8	46.9	48.3	55.9	70.1	18.8
2%	36.3	20.3	53.0	54.5	63.0	78.8	21.5
1%	41.1	23.1	59.9	61.5	71.1	88.9	24.4
0.5%	46.8	26.5	67.9	69.7	80.4	100	28.0
0.1%	60.9	35.2	87.2	89.5	103	127	37.1

*Wood used 117.2 km^2 for design based on the ArcticDTM.

5.2 Probable Maximum Flood Approximation

5.2.1 PMF Approximation Using the Creager Approach

A well-known flood envelope diagram is the Creager diagram as published in Creager et al. (1945). Creager plotted a large number of "unusual" flood discharges from rivers in the USA and a number of other countries to produce a curved band of data on double-logarithmic paper. The higher data points in the Creager diagram represent basins with the greatest flood peak potential, i.e., basins experiencing severe rainfall or characterized relatively steep slopes, compact drainage networks, relatively impervious surfaces and little storage (Alberta Transportation 2004). An empirical formula was developed with varying values of the Creager coefficient, C. Higher values of C represent higher degrees of flood severity. Most data are bounded by an upper limit of C = 100. Neill (1985) found that Canadian data appear to follow the trend of Creager curves fairly well (though it would be possible to fit a straighter log-log curve) and that most values lie between C=20 and C=45. Alberta Transportation (2004) states that to 2003, a value of 45.0 was the largest C value recorded in Canada (Station 07GF001 Simonette River near Godin). Lawford et al. (1995) state that when floods are viewed in a global context, Canadian floods tend to be less severe than those found in some other parts of the world.

The Creager formula (in SI units) is:

$$Q = 1.303\,C\left(\left(\frac{A}{2.59}\right)^{\frac{0.936}{A^{0.048}}}\right)$$

Where Q = unusual flood peak discharge in m^3/s,
C = the Creager coefficient, and
A = the drainage basin area in km^2.

Using available hydrometric records for 61 WSC hydrometric stations within the Yukon River basin in Canada, values of C are calculated for the highest observed annual peak discharge. The basin areas of these stations range from 76.9 km^2 to 288 000 km^2 and the number of recorded annual peak flows from 4 to 56 years. Existing hydrometric records result in Creager values less than 30. The highest C value is 24.2 for the hydrometric station 09ED0001 Yukon River at Eagle (drainage basin area = 288 000 km^2; 24 recorded annual peak discharges) and the second highest value of C is 21.3 is for 09AA012 Wheaton River near Carcross (drainage basin area = 864 km^2; 56 recorded annual peak discharges). Most (77%) of the Creager values determined for 61 hydrometric stations are less than 10.

For preliminary design purposes, the probable maximum flood (PMF) is assumed to have a magnitude approximately equal to the value determined using a Creager coefficient of 40. This results in a PMF of 891 m^3/s (or 7.6 $m^3/s/km^2$). This value compares well with other extreme floods in Canada noted by Neill (1985) for basins of a similar size at Rainy River, British Columbia, 1958 (6.17 $m^3/s/km^2$) and Norrish Creek, British Columbia, 1984 (4.27 $m^3/s/km^2$). Though these watercourses are both in a difference hydrological setting the data does show the Creager based calculation for Clinton Creek is a reasonable first estimate.

5.3 CDA Design Flows

For a significant consequence dam, the design flood flow is specified under the CDA Dam Safety Guidelines for Closure – Passive Care as 1/3 between the 1000-year return period flood flow and PMF.

Table 5-3: Design Flood Flows by CDA Consequence Class

Consequence Class	Flood Flows	Clinton Creek at Hudgeon Lake Outlet, m3/s
Low	1/1000	90
Significant	**1/3 between 1000-year and PMF**	**357**
High	2/3 between 1000-year and PMF	624
Very High/Extreme	PMF	891

An inflow hydrograph for the IDF (1/3 between the 0.1% AEP flood flow and PMF) was created using HEC_HMS. Since the interest is primarily interested in peak flows, an assumption was made that the storm would last approximately 24 hrs. The hydrograph is shown in **Figure 5-3** below.

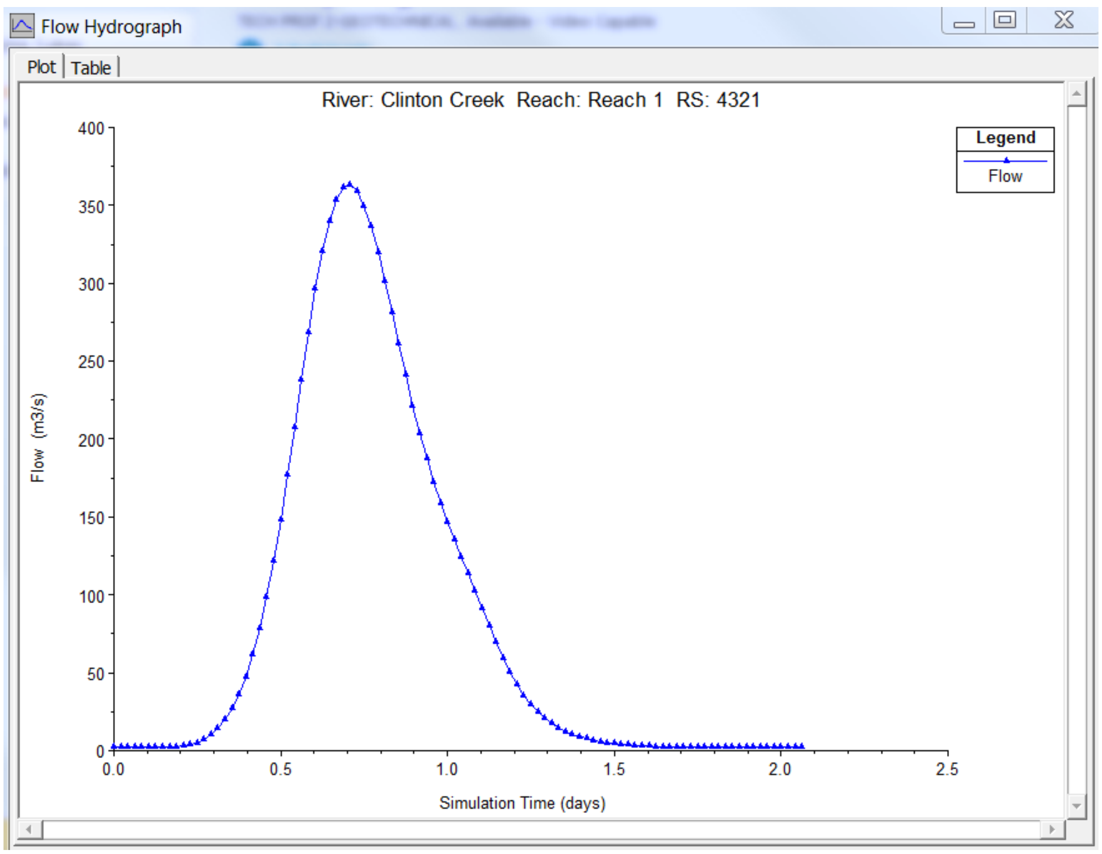


Figure 5-3: IDF Design Hydrograph

6.0 Dam Breach Hydraulic Modelling

6.1 Software

Hydraulic modelling of the dam breach scenarios was undertaken using US Army Corps of Engineers HEC-RAS v5.0.6. The HEC-RAS system includes river analysis components for: (1) steady flow water surface profile computation; (2) one- and two-dimensional unsteady flow simulation; (3) movable boundary sediment transport computation; and (4) water quality analysis. As a key feature, all four components use common geometric data representation and geometric and hydraulic computation routines. In addition to these river analysis components, the system offers several hydraulic design features that can be invoked once basic water surface profiles are computed. It is possible to model dam breaches using an inline weir structure. As an expansive floodplain is not present, a 1-D model was deemed appropriate for modelling the various breach scenarios.

6.2 Model Extents

The hydraulic model extents run from the upstream limit of Hudgeon Lake to the Yukon River as shown in **Figure 6-1**. Interpolated cross-sections within the model are not shown for figure clarity. The downstream boundary was set to a normal depth in the Yukon River with gradient of 0.0003 m/m.

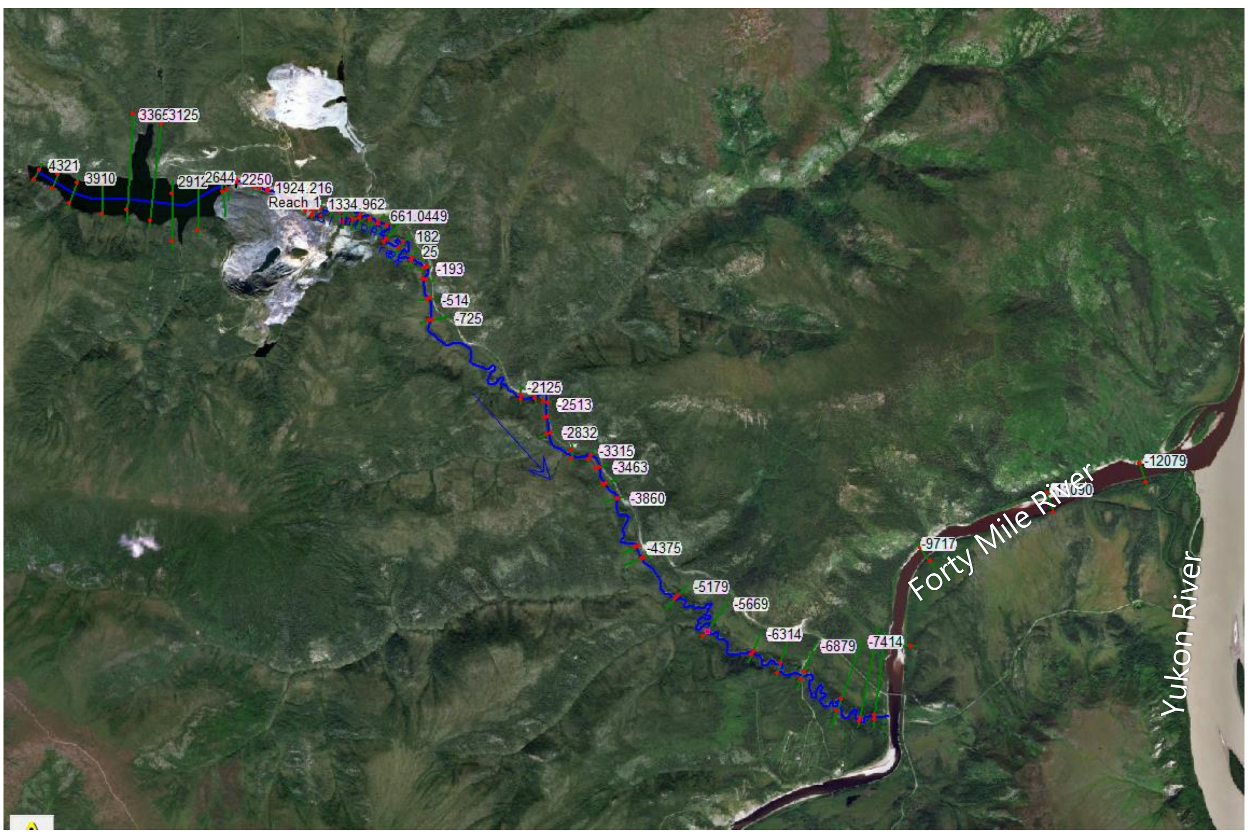


Figure 6-1: Hydraulic Model Extents and Cross-Section Locations

6.3 Dam Breach Methodology

A dam breach assessment must evaluate the incremental consequences of a breach. As such, the model employed must extend sufficiently downstream to a point where the breach has little noticeable impact on water levels. For this study, the model extent ran from the inflow to Hudgeon Lake to the Yukon River.

Since the goal of the assessment is to evaluate incremental consequences of a breach, the model was first run using the IDF with no breach of the drop structures. The model was then run with the same geometry and inflow but with a breach of the impoundment. It was assumed that the breach had occurred at the drop structures. The drop structures were modelled as a single inline spill unit set to breach when the flow from the drop structures exceeded 90 m^3/s (the 0.1% AEP flow) as discussed in the following paragraph.

The breach mechanism at Clinton Creek is unique and it is difficult to quantify the breach depth, width and formation time with any degree of certainty. For this reason, a number of scenarios were modelled to test the sensitivity of the model to varying dam breach parameters.

6.3.1 Breach Width and Time of Formation

As a starting point, parameters defining the overtopping dam breach were estimated using the method developed by Froehlich (Froehlich, 2008). Using Froehlich's equations and assuming a breach elevation level of 399.68 m (the 1% slope breach depth of 12.2 m), the calculated average breach width was measured as 70 m with a time of formation of 1.3 hrs. The bottom width (needed for HEC-RAS) depends on side slopes, and an assumption was made that the sides would be approximately 2H:1V. These numbers (70 m average width and 1.3 hrs of formation time) correlate reasonably well with those calculated by UMA in 2000 (average breach width of 80 m and 0.8 hrs of formation time).

The calculated breach formation time was adjusted to account for erosion protection measures in place at the site (gabions, articulated concrete mats and riprap placed at the bottom of DS4). A reasonable assumption was made that the time of formation would be in the order of 2 hrs.

Another important consideration concerns the timing of breach initiation. Design standards for the gabion drop structures were not available for this study. In 2010, a major flood perhaps in the order of 1% AEP or higher was experienced, causing significant damage to DS4. However, the drop structures did not fail completely. An assumption was therefore made that the gabions would be able to withstand up to a 1-in-1000-year flood event flow. When the IDF reached the 0.1% AEP discharge level (90 m^3/s), the breach was initiated.

6.3.2 Breach Progression

It is expected that the breach of the gabion drop structures would not follow a linear progression; rather, the breach would progress at a slower rate until such time as the rock held within the gabions has eroded and the underlying waste material has been exposed, leading to rapid downcutting and erosion to the predicted breach elevation. **Figure 6-2** presents the assumed non-linear progression. Note that the progression time is non-dimensional, and the curve is applied to the varying breach formation times shown in **Table 6-1** below.

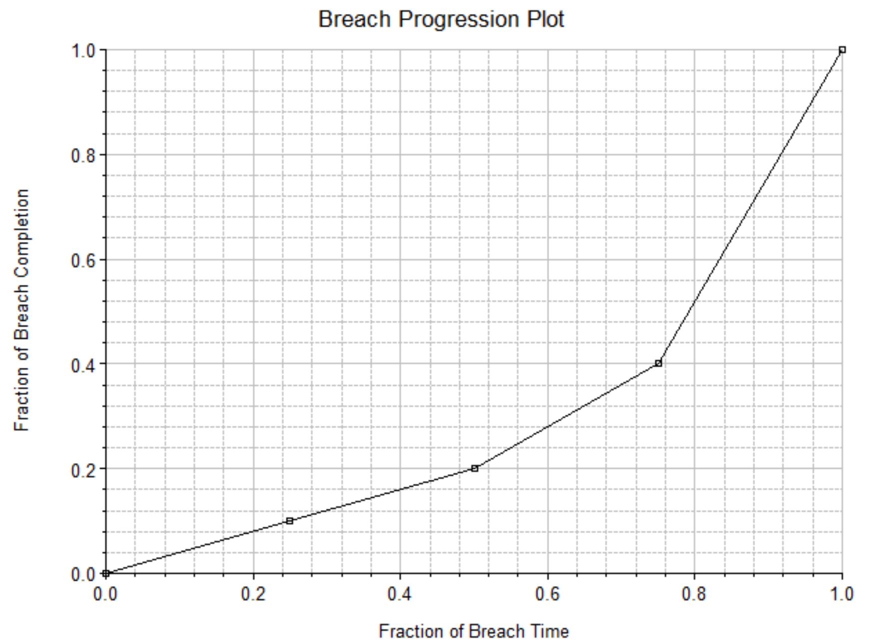


Figure 6-2: Assumed Breach Progression for Hydraulic Modelling

6.3.3 Sensitivity Analysis

Since the breach depth, width and formation time are difficult to predict with any certainty, a sensitivity analysis was conducted to determine if the parameters would significantly affect the findings of this study. In turn, various breach scenarios were analyzed as shown in **Table 6-1**.

Table 6-1: Clinton Creek Breach Scenarios – Flood-Induced Failure

Scenario	Breach Elevation (m)	Breach Width (m)	Breach Time of Formation (hrs)	Peak Discharge Downstream from Breach (m^3/s)
1	399.68	70	2	1131
2	399.68	70	4	1002
3	399.68	70	6	911
4	399.68	40	2	1074
5	399.68	40	4	938
6	399.68	40	6	856
7	401.91	70	2	1131

From **Table 6-1** above, it is evident that the breach elevation and width are not sensitive parameters. The breach formation time, however, does make quite a significant difference to the flood flows. It is intuitive that a shorter breach time will increase the peak discharge; however, with a longer breach duration, the peak of the inflow hydrograph matches the peak breach flow more closely; thus, the difference is not as significant as might be expected.

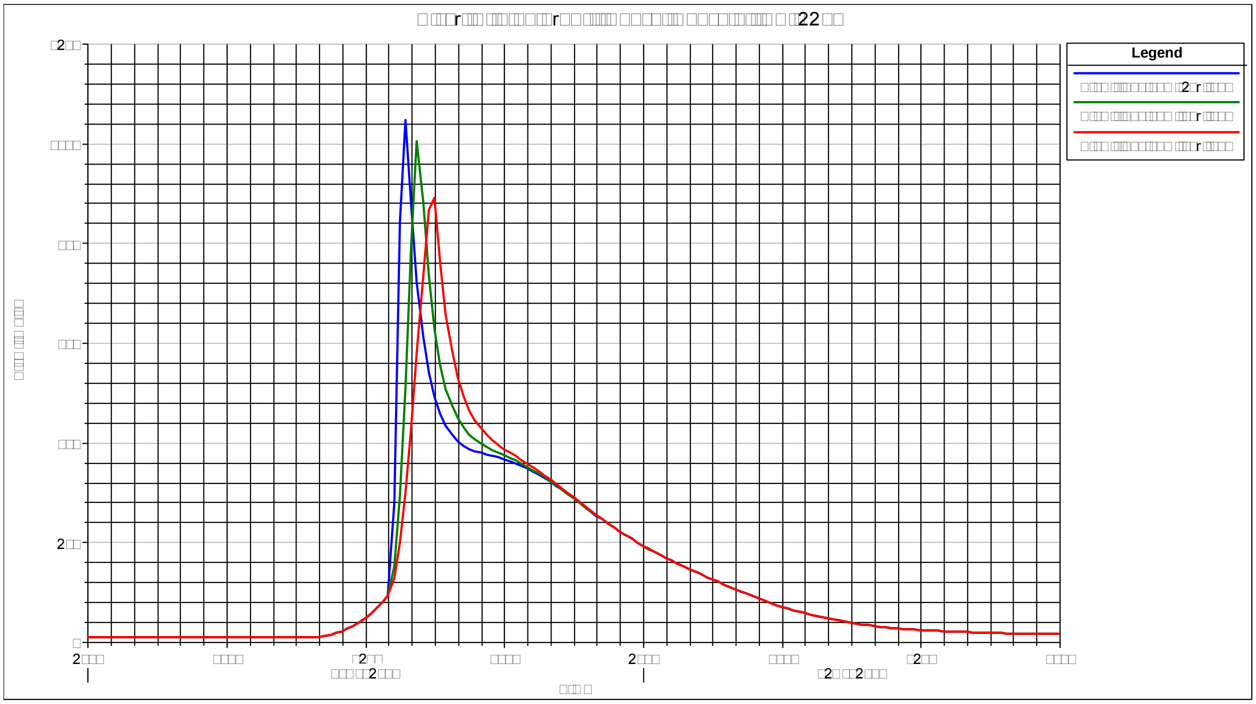


Figure 6-3: Flood-Induced Breach Hydrographs for Various Times of Formation

6.3.4 Breach Hydrographs

As the outflow is most sensitive to breach formation time, representative outflow hydrographs showing comparisons between breach formation times are presented in **Figure 6-3**. The hydrograph is characterized by a very steep rising limb reaching a maximum discharge level of 1131 m^3/s for the 2-hr breach formation time. As one would expect, the peak of the breach hydrograph is dominated by the outflow from the lake. As such, the incremental discharge is very significant. As the limb falls, the incremental flood flows reduce and the flood is dominated by the IDF.

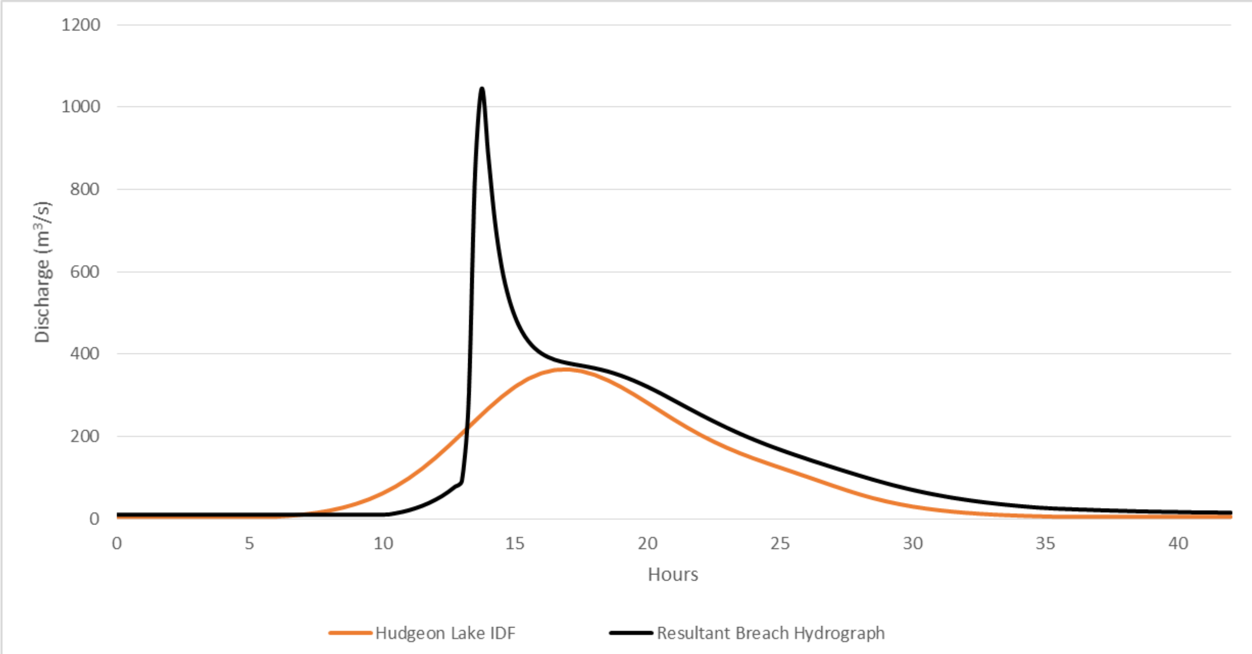


Figure 6-4: Plot Showing the IDF and Resultant Breach Hydrograph

6.3.5 Comparison with Empirical Methods

The adopted design model was compared with the empirical results given in Section 3.1.2 for the 1% and 2% gradient headcut (from the toe of DS4) scenarios.

Table 6-2: Comparison of Modelled Versus Calculated Flood Discharges

Headcut Slope from Toe of DS4	Breach Elevation (m)	HEC-RAS Breach Model (m^3/s)	Empirical Equation (m^3/s)
1%	399.68	1131	1214
2%	401.91	1131	1051

A key assumption affecting the HEC-RAS modelled flood peak was the timing of the breach in relation to the inflow hydrograph. The resultant flood peaks would have been higher had the timing of the breach been synchronized with the peak of the inflow hydrograph. However, there is no real basis for artificially synchronising these events given the susceptibility of the drop structures to flood events of a lesser magnitude. The assumption made, as described in Section 6.3.1, is considered to be adequately conservative for the purposes of the 10% design phase. It should also be noted that these breach flood discharges are approximately double those estimated by UMA (2000) and as such are considered adequately conservative.

6.4 Data Availability

The model was constructed from available satellite-based digital terrain model information provided by the Yukon Government. Downstream from the confluence with Wolverine Creek, the only data available for the analysis were ArcticDTM data produced by the Polar Geospatial Centre at the University of

Minnesota. When the data were downloaded into the model, it became apparent that vertical datums for the two datasets differed by approximately 10 m (the ArcticDTM data averaged 10 m above those of the dataset provided by YG), and hence an adjustment of -10 m was made to the ArcticDTM. This was deemed appropriate because the absolute levels downstream from the site are not important; rather, the analysis focused on the incremental change in flood levels.

6.5 Flood-Induced Failure

6.5.1 Results

Figures 6-5 and **6-6** demonstrate how flow is attenuated as the hydrograph passes down Clinton Creek through the Fortymile River to discharge into the Yukon River. From the initial breaching of the drop structures, the hydrograph takes approximately 2.5 hrs to pass downstream to the Yukon River with the flow reducing from a peak of 1,131 m^3/s just downstream from Hudgeon Lake to approximately 900 m^3/s at the Fortymile River and to 560 m^3/s once it reaches the Yukon River.

Figure 6-6 presents a profile of the maximum flow along the modelled reach. It demonstrates that Hudgeon Lake attenuates the IDF inflow from 357 m^3/s to approximately 220 m^3/s; however, after the breach occurs, the flow quickly rises to a maximum of 1,131 m^3/s.

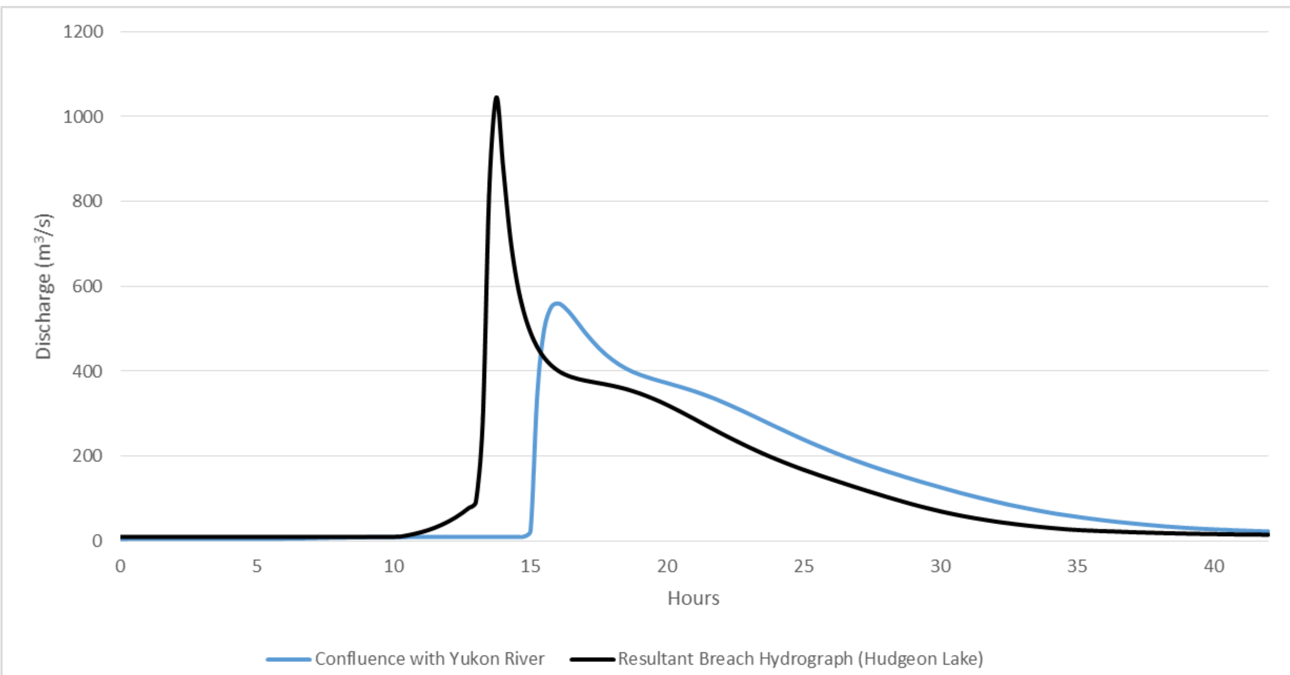


Figure 6-5: Flow Attenuation from Hudgeon Lake to the Yukon River

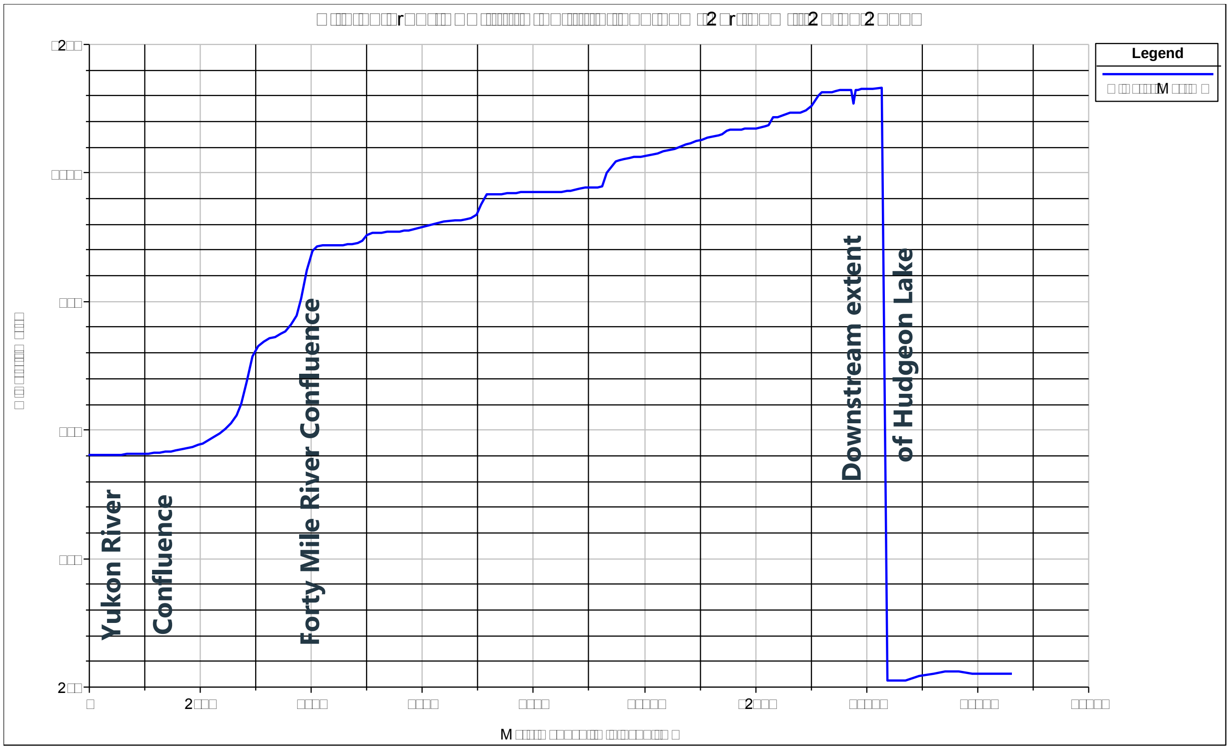


Figure 6-6: Flow Attenuation from Hudgeon Lake to the Yukon River

Figures 6-7 to **6-11** below show the incremental peak water levels for the flood-induced breach scenario. Immediately downstream from DS4, the flood-induced breach water levels are approximately 5.03 m higher than when the structure does not breach. By the time the flood reaches the Yukon River, the incremental rise in water levels is only 1.42 m higher. This is attributable to the significant attenuation of the flood hydrograph as it passes through the system and to the cross-sectional properties of the Yukon River (it is approximately 500 m wide). This 1.42 m incremental change in water level will be contained within the banks of the Yukon River. Given the improbability of extreme flooding in the Yukon River occurring at the same time as a dam breach in Clinton Creek, it can be concluded that the adverse effects of a dam breach at Clinton Creek would dissipate by the time a flood reaches the Yukon River. Therefore, the selected downstream boundary at the Yukon River is considered acceptable.

6.5.2 Breach Hazard

Figure 6-12 below shows the average cross-sectional channel velocity resulting from a flood-induced breach with a 1% headcut, 2-hr breach formation time and 70-m width. The plot shows that immediately downstream from Hudgeon Lake, average channel velocities can be expected to reach close to 10 m/s; however, as the flood progresses downstream, velocities of less than 5 m/s are typical until the Fortymile River is reached, where velocities will drop to less than 1 m/s. It can therefore be concluded that in the event of a breach, a very strong hazard is posed to populations within the floodway. A flood wave takes approximately 2-2.5 hrs to travel from the Hudgeon Lake outlet to the Fortymile River.

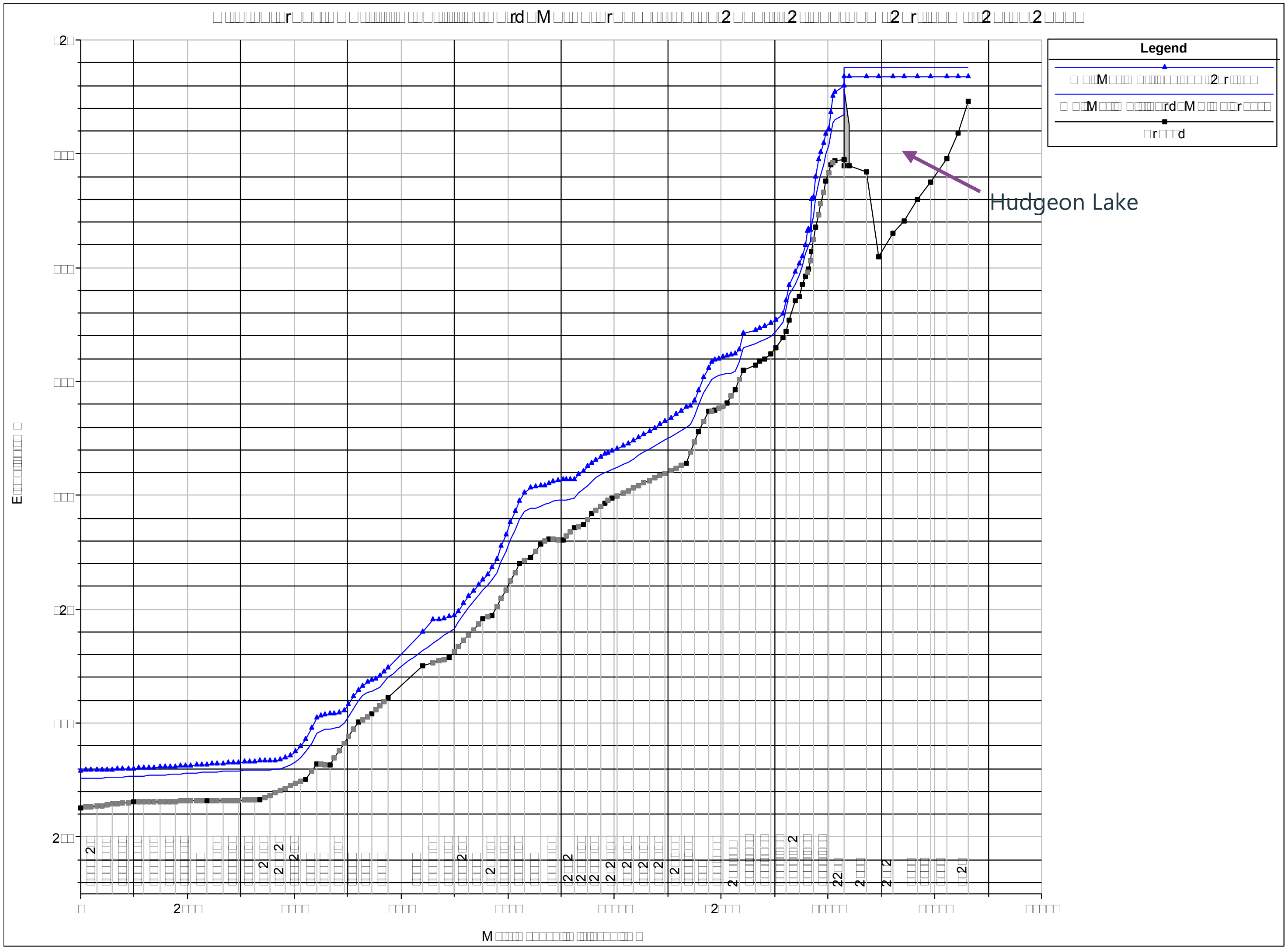


Figure 6-7: Profile of Peak Modelled Water Levels – Flood-Induced Breach

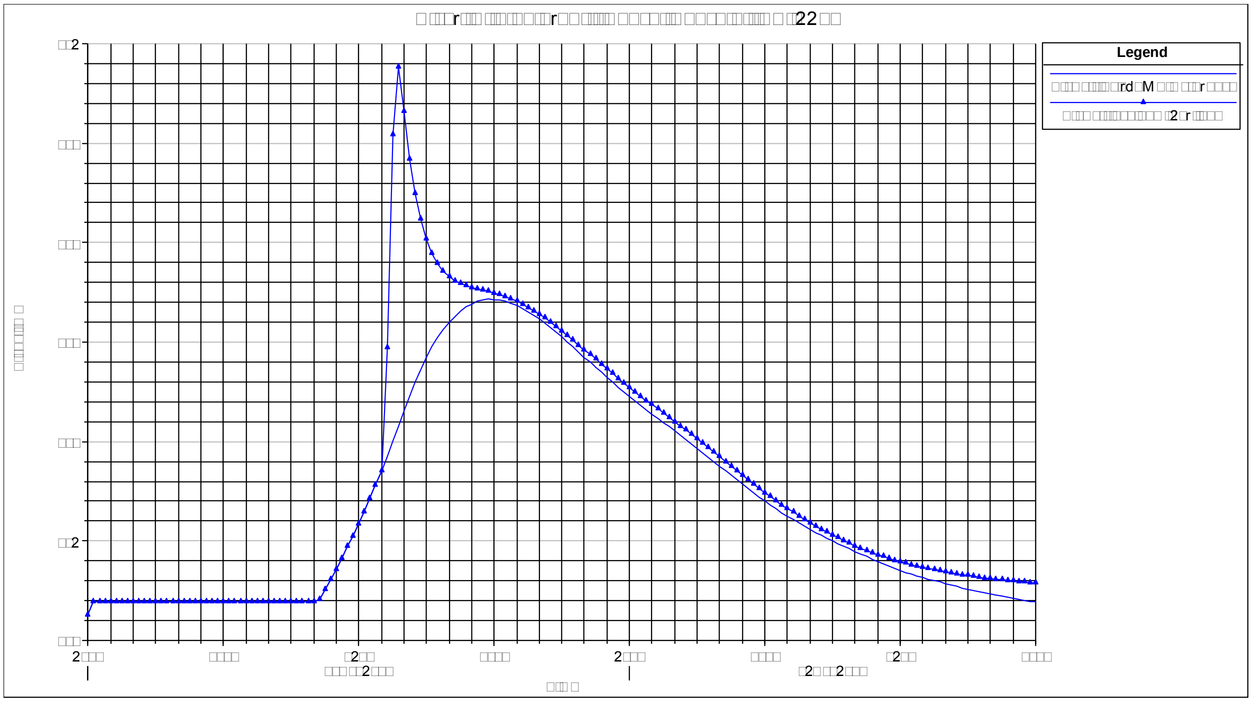


Figure 6-8: Stage Hydrograph Comparing Flood-Induced Breach and Base Case Immediately Downstream from DS4

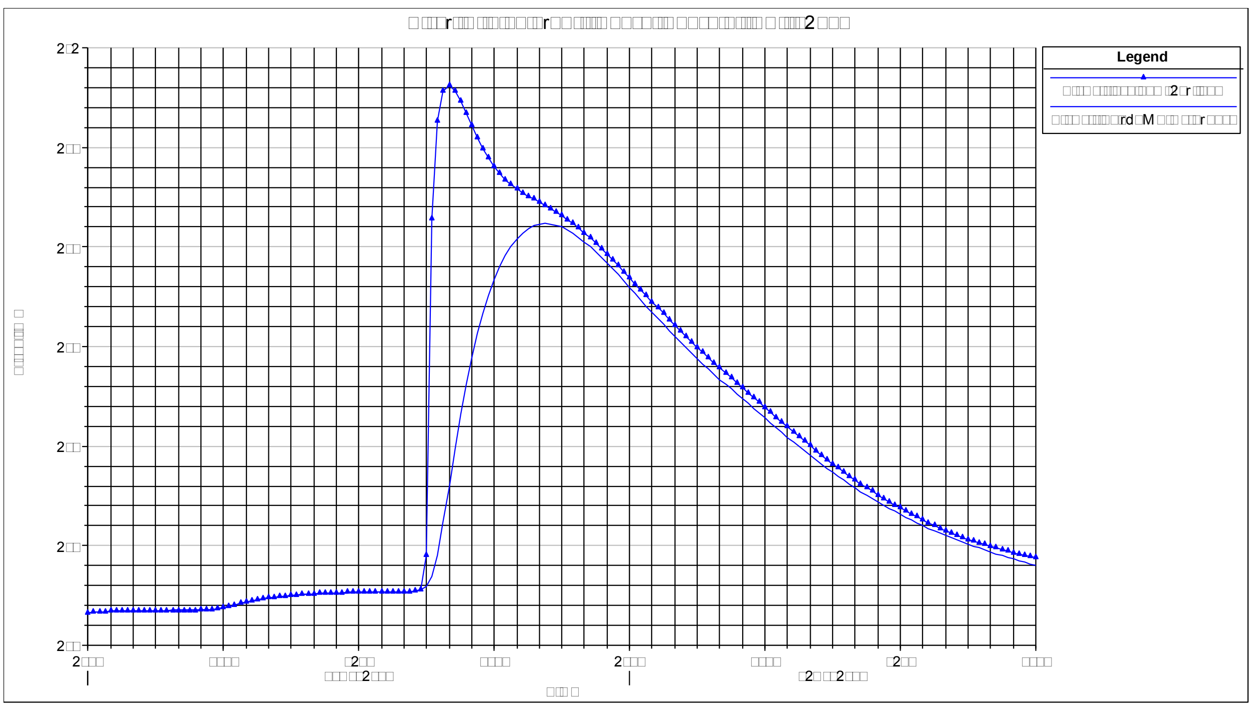


Figure 6-9: Stage Hydrograph Comparing Flood-Induced Breach and Base Case at the Yukon River Confluence

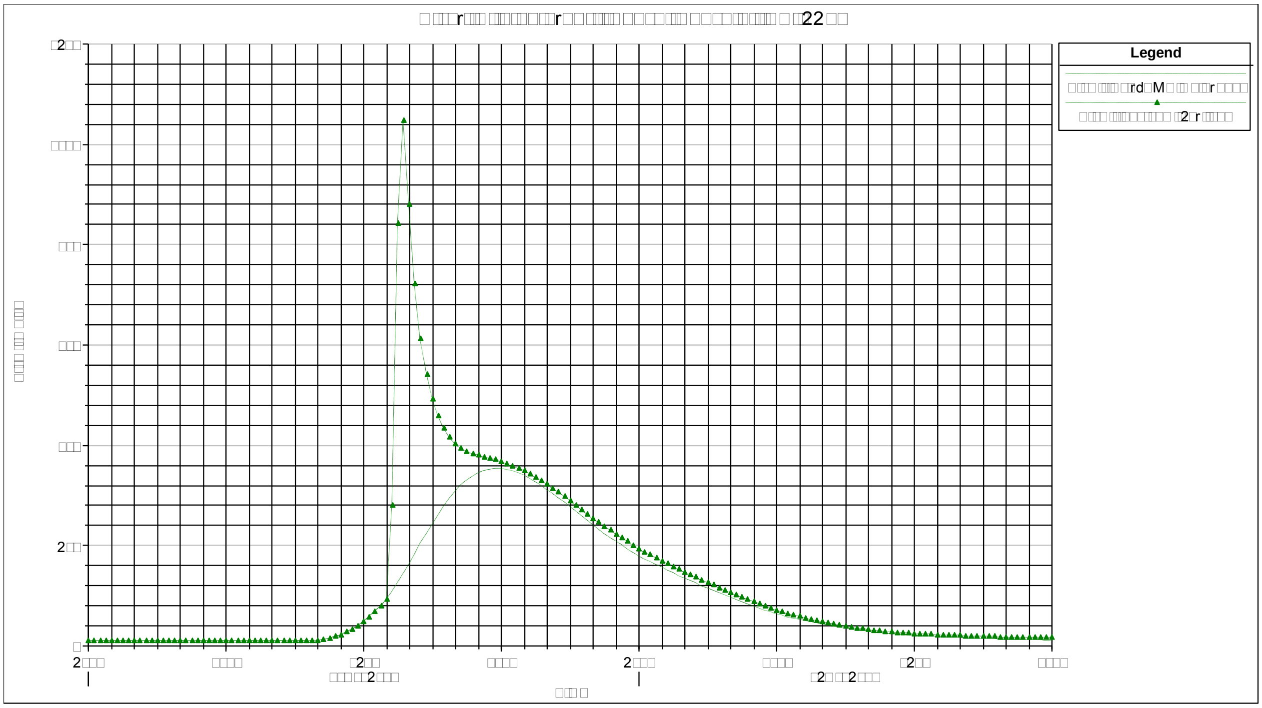


Figure 6-10: Flow Hydrograph Comparing Flood-Induced Breach and Base Case Immediately Downstream from DS4

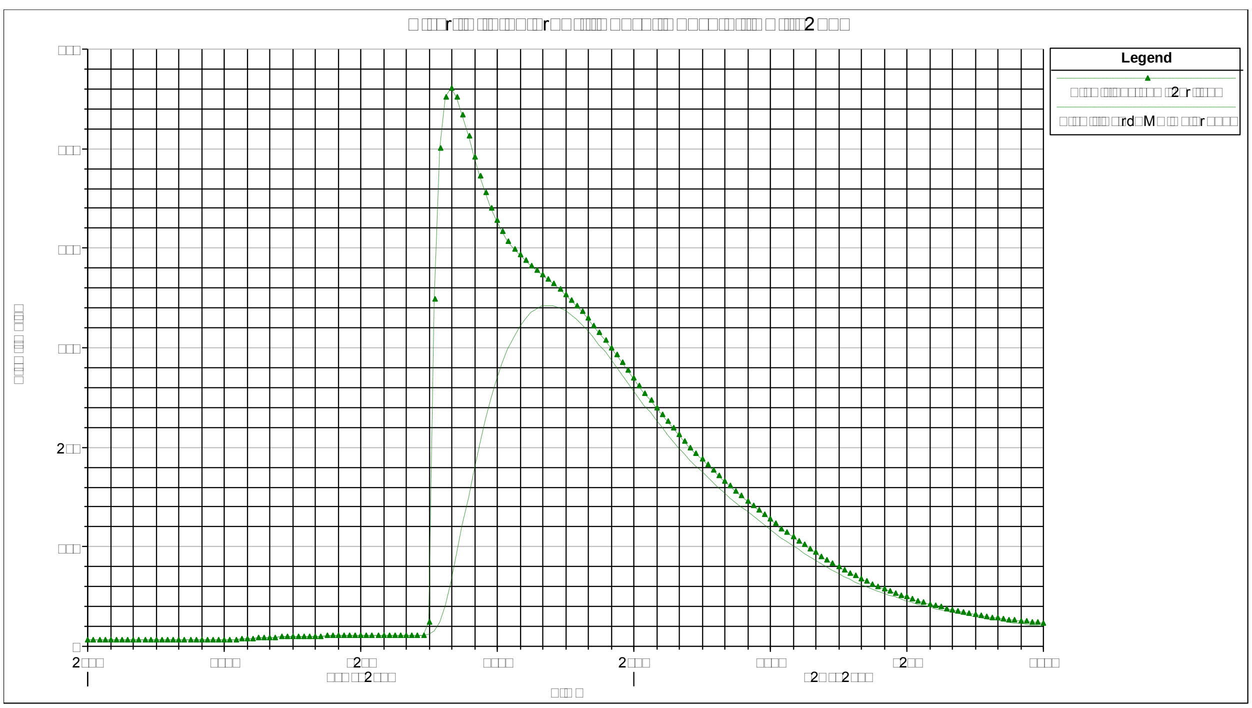


Figure 6-11: Flow Hydrograph Comparing Flood-Induced Breach and Base Case at the Yukon River Confluence

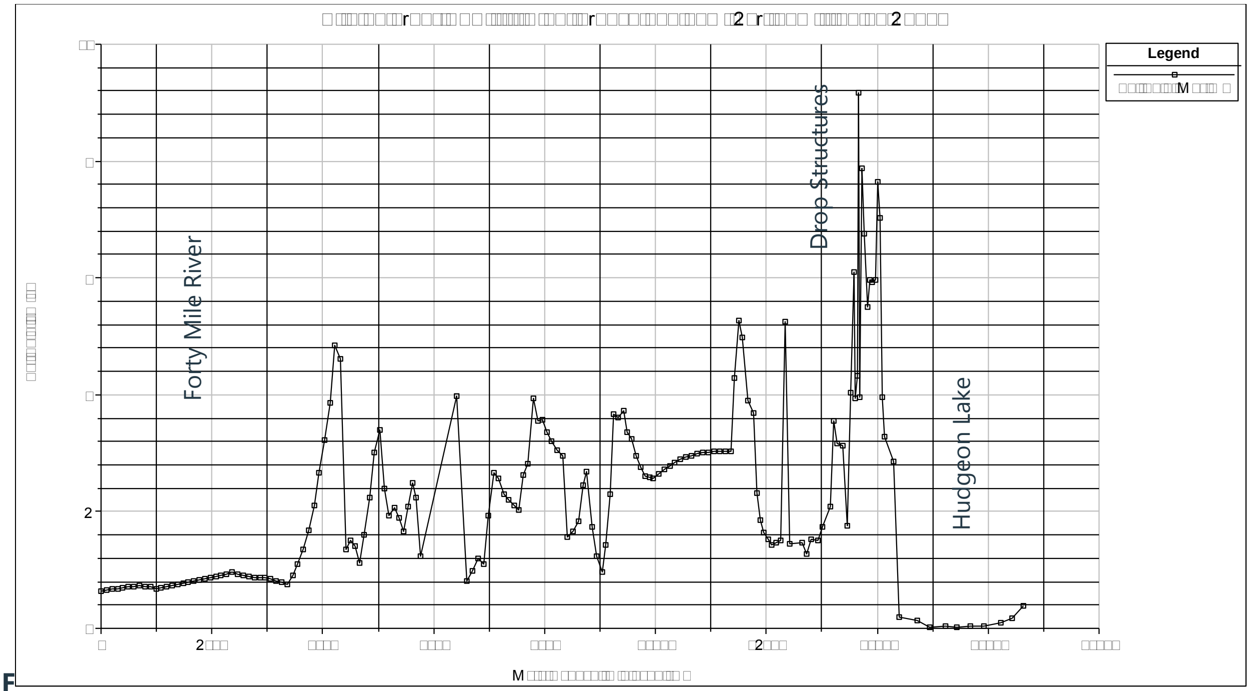


Figure 6-12: Average Channel Velocity Resulting from a Breach

6.6 Sunny Day Failure

A standard part of dam breach assessment involves assessing the consequences of a “sunny day” failure of a given structure. This assumes that a flood is initiated by some other mechanism such as piping. Since the purpose of this work was to inform the criteria for 10% design, a sunny day failure analysis was not considered necessary to achieve the overall project goal.

7.0 CC2 Regime Channel Sediment Transport and Breach Modelling

7.1 Model Introduction

The HEC RAS model was developed to simulate the regime channel (CC2) for Clinton Creek under storm events including the 1% AEP (1 in 100 years), 0.1% AEP (1 in 1000 year) and, to test a more extreme case, the 1/3 between 0.1% AEP and PMF. For each storm event, a sediment transport model was run to demonstrate consequences of channel failure and the effectiveness of channel protections. The following two conditions were tested:

1) an unarmoured channel to test the maximum potential erosion of the waste rock using the observed rock gradation from the 2016 and 2018 geotechnical field investigation (**Figure 1-5**); and
2) a riprap protected channel per the CC2 design.

7.2 Background on HEC RAS Sediment Transport Functions

HEC-RAS computes sediment transport based on sediment size and channel flow velocity, and hence sediment characteristics and flow data are the two most important parameters for modelling sediment transport. Flow data can be entered as either unsteady or quasi-unsteady flow. While the unsteady flow option visualizes the flow continuously as a smooth hydrograph, the quasi-unsteady flow option breaks it down into a series of steady flows similar to a histograph. Although the unsteady flow option is preferable as it conserves flow and accounts for storage, it is often unstable. Therefore, the quasi-unsteady flow option was adopted for this study. The sediment data file defines characteristics including grain size, specific gravity, Manning's roughness (n) and cohesion values. For Clinton Creek, a specific gravity value of 2.65, Manning's roughness coefficient of 0.03 and cohesion of 0 are set as the default as recommended in the HEC-RAS manual, and the material gradation (grain size) is discussed in the following section.

In cases where channel erosion is bounded by a non-erodible boundary, the maximum erosion depth is limited. However, this non-erodible boundary exists only deep below the waste rock in Clinton Creek, and therefore the potential for erosion is not limited by this non-erodible boundary. The maximum depth was set below any conceivable down cutting range. In addition, a movable bed limit in the horizontal direction must be entered to eliminate unreasonable erosion such as that occurring above the water surface. In this model, the moveable limit was set to the maximum flood width based on the assumption that there would be no channel avulsion and that a flood would erode the existing banks.

7.3 Data Description

Assuming no sediment inflow at the upstream boundary, the model corroborates grain sizes plotted in **Figure 1-5** with a maximum erosion width of 35 m (approximately the peak flood width) and a side slope of 1:1. The regime channel is assigned natural sediment for the first scenario and riprap for the second while material in the downstream area remains the same for both scenarios. Riprap gradation specifications adopted were taken from the BC Ministry of Environment's riprap guide. For modelling purposes, the change in gradation shown in drawings for CC2 (especially at the outlet apron) has been disregarded, as it is inconsequential to this modelling work. The riprap has the gradation shown in **Table 7-1** below.

Table 7-1: Riprap Gradation for Regime Channel

Riprap Class	Nominal Thickness of Riprap (mm)	Rock Gradation Percentage Greater than the Given Rock Mass (kg)		
		85%	50%	15%
25 kg	450	2.5	25	75

A hydrograph of storm events simulated is plotted in **Figure 7-1**. The 1% AEP flood reaches a peak discharge level of 61.8 m^3/s, the 0.1% AEP flood reaches a peak discharge level of 89.8 m^3/s and the 1/3 between 0.1% and PMF (denoted in the table as 1/3 PMF) has a peak flow of 357 m^3/s.

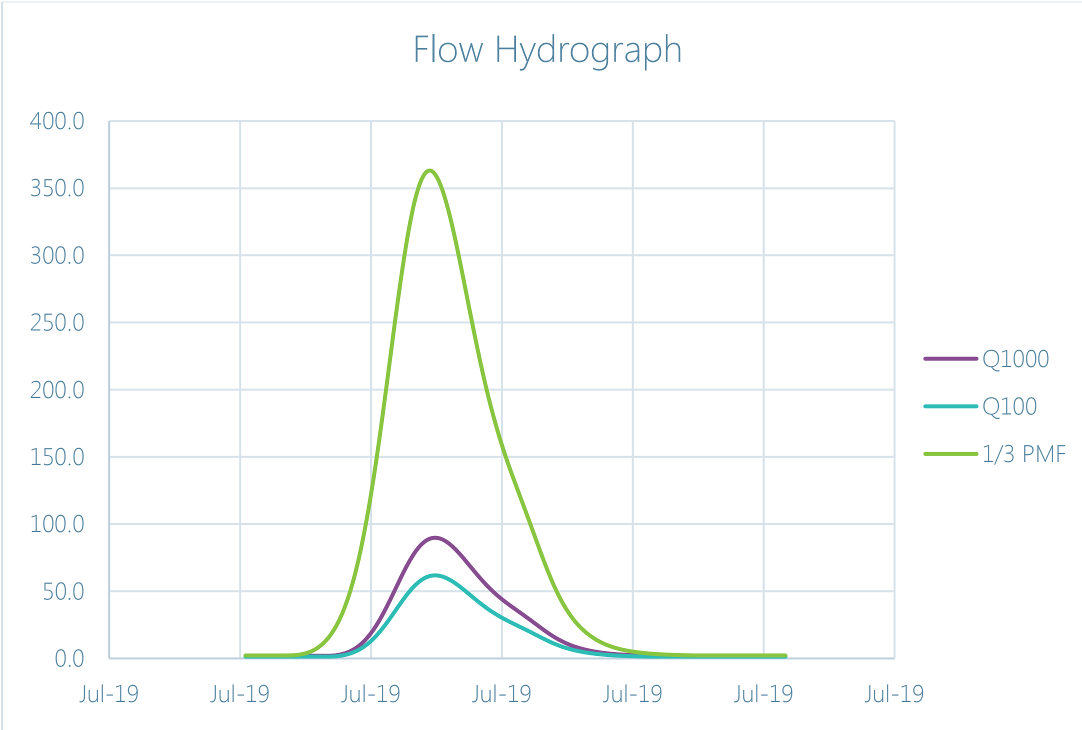


Figure 7-1: Flow Hydrograph of a Simulated Storm Event

7.4 Discussion of Sediment Transport Model Results

The model results show that the unprotected channel would erode 2.6 m in the event of a 1% AEP flood, 3.3 m in the event of a 0.1% AEP flood and 7.1 m in a 1/3 between 0.1% AEP and PMF while no erosion would occur during any of these events with riprap protection. The model demonstrates that the selected riprap size is adequate to resist the designed flows up to and including the 1/3 between 0.1% AEP and PMF.

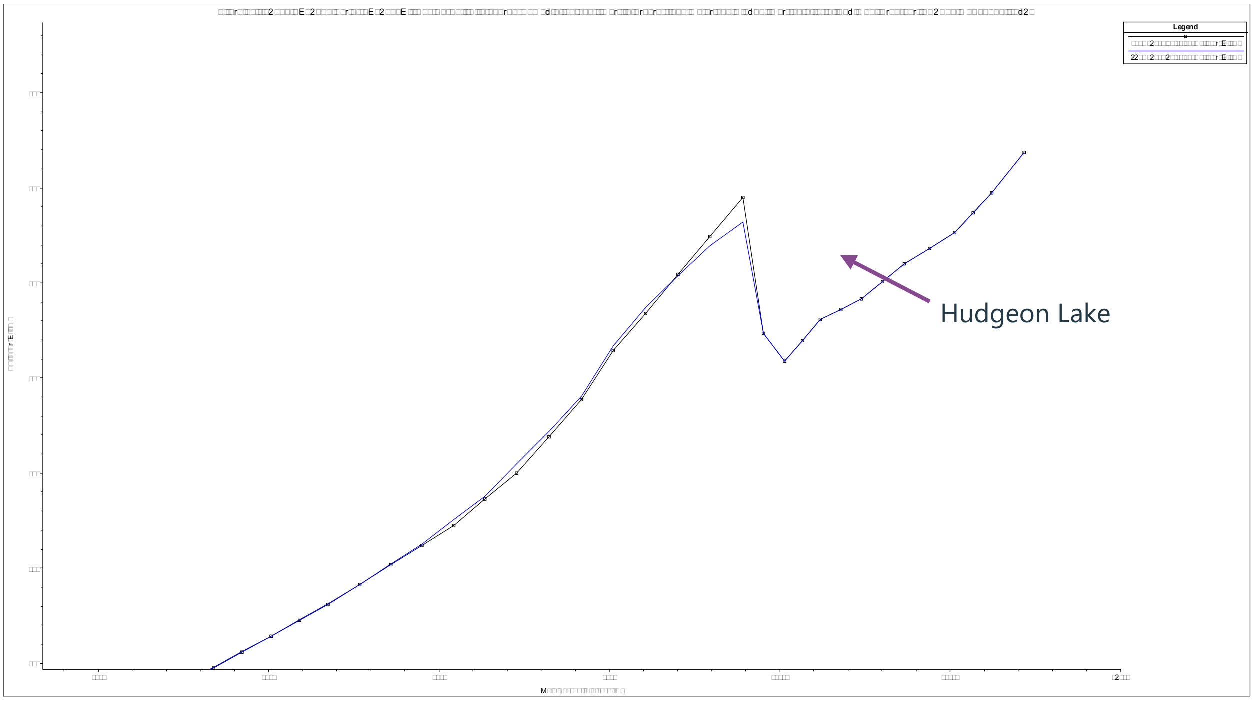


Figure 7-2a: Profile of Sediment Transport During a 1% AEP Flood – No Channel Armouring

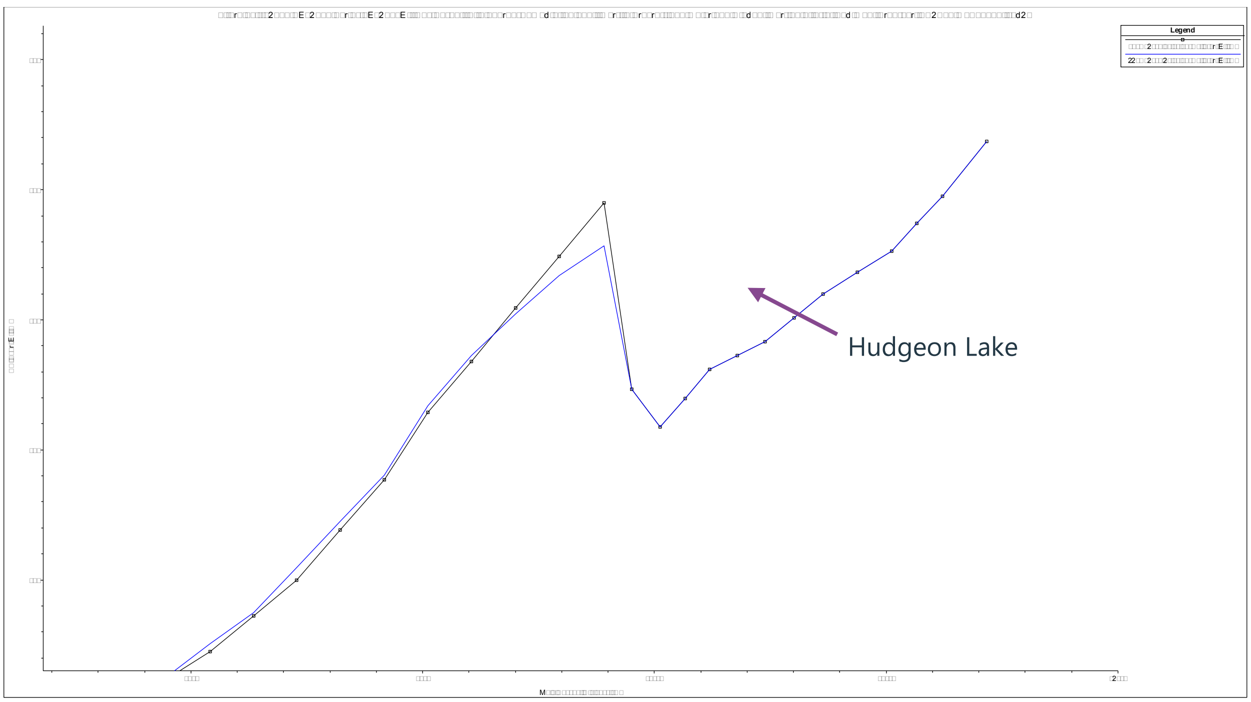


Figure 7-2b: Profile of Sediment Transport During a 0.1% AEP Flood – No Channel Armouring

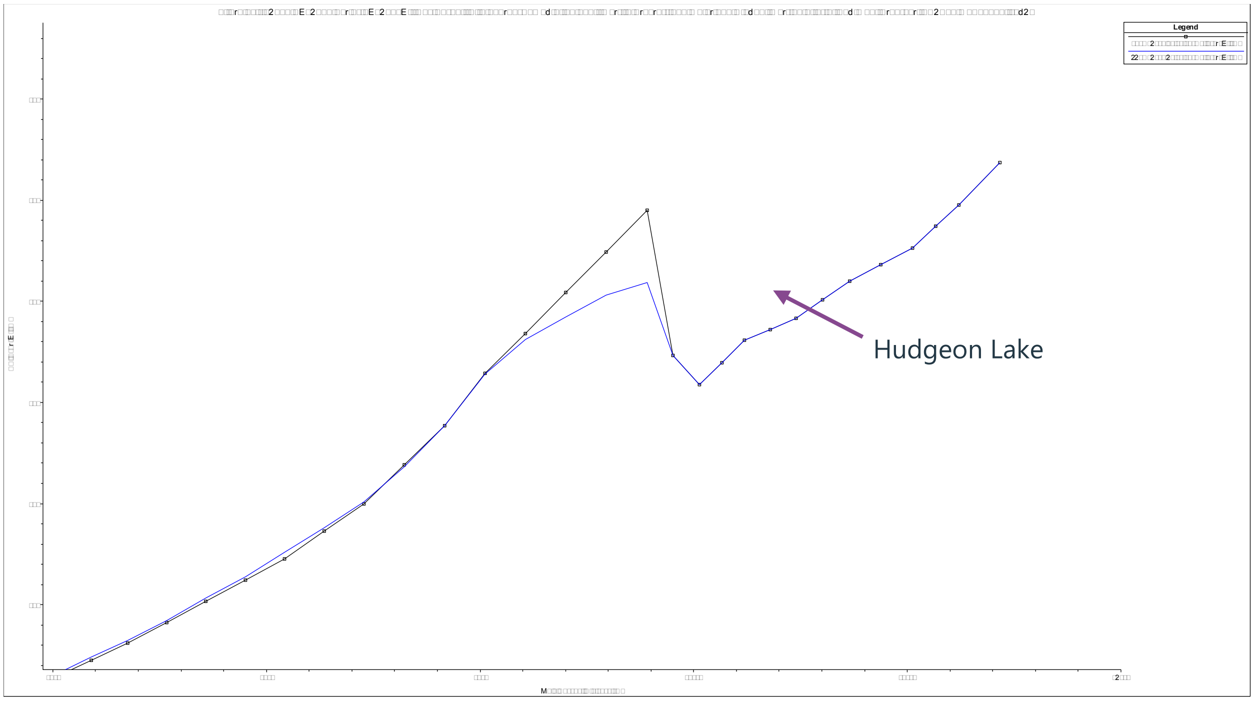


Figure 7-2c: Profile of Sediment Transport during an IDF Flood – No Channel Armouring

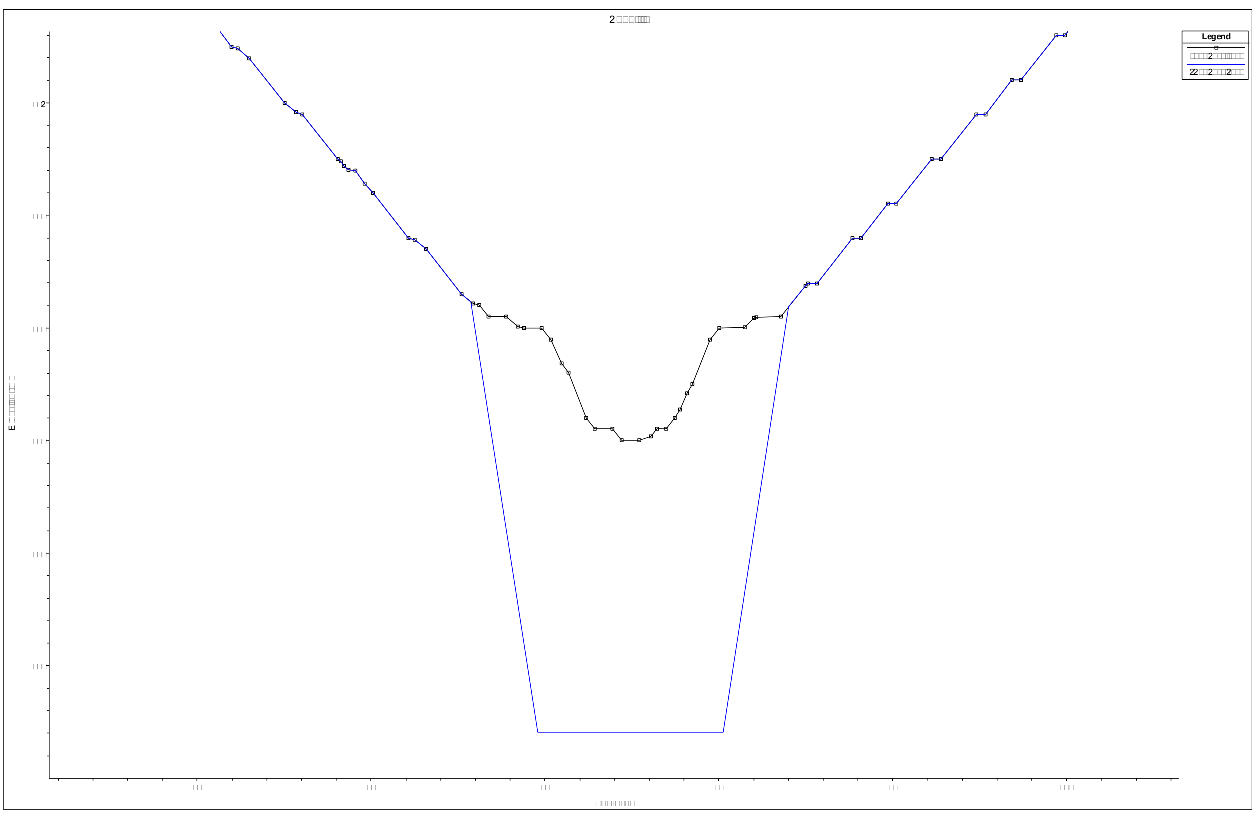


Figure 7-2d: Downcutting Cross-Section of Channel with Natural Sediment during a 1% AEP Flood

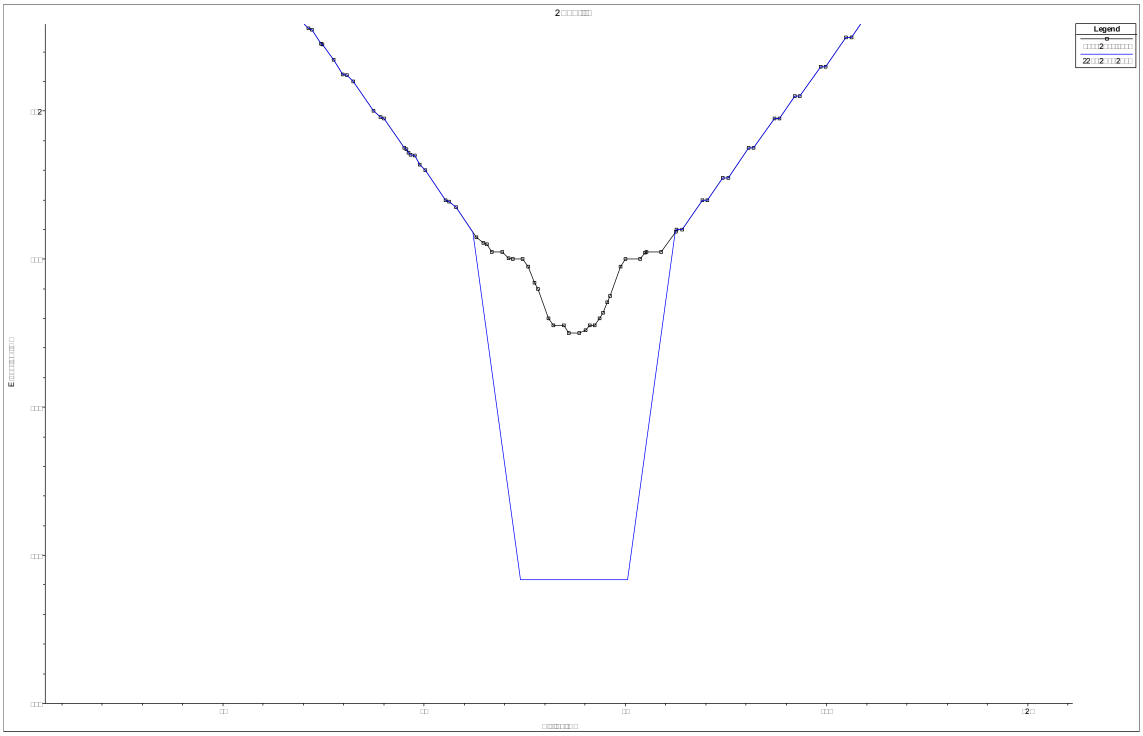


Figure 7-2e: Downcutting Cross-Section of Channel with Natural Sediment during a 0.1% AEP Flood

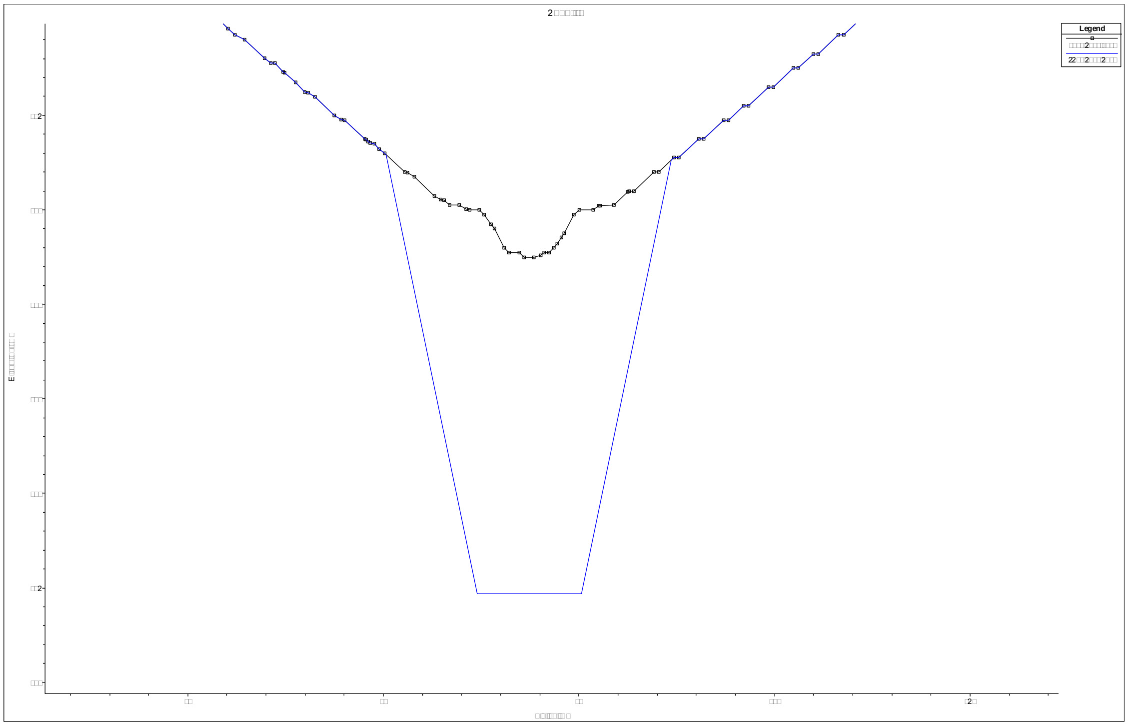


Figure 7-2f: Downcutting Cross-Section of Channel with Natural Sediment with Discharge 1/3 between a 0.1% AEP Flood and the PMF

7.5 Regime Channel Breach Modelling

The quasi-unsteady sediment transport model was used to generate a downcutting time-series which in turn enabled breach modelling for the regime channel. The time series shows the downcutting rate to be approximately 0.23 m/hr for the IDF lasting over 30 hrs. Conservative assumptions were made regarding the breach conditions. These were:

- downcutting occurred over 12 hours (compared to the 30+ hrs shown in Figure 7-3);
- the channel was unarmoured and it was waste rock (see Fig 2-5) being eroded; and
- the IDF for a Significant class dam was adopted.

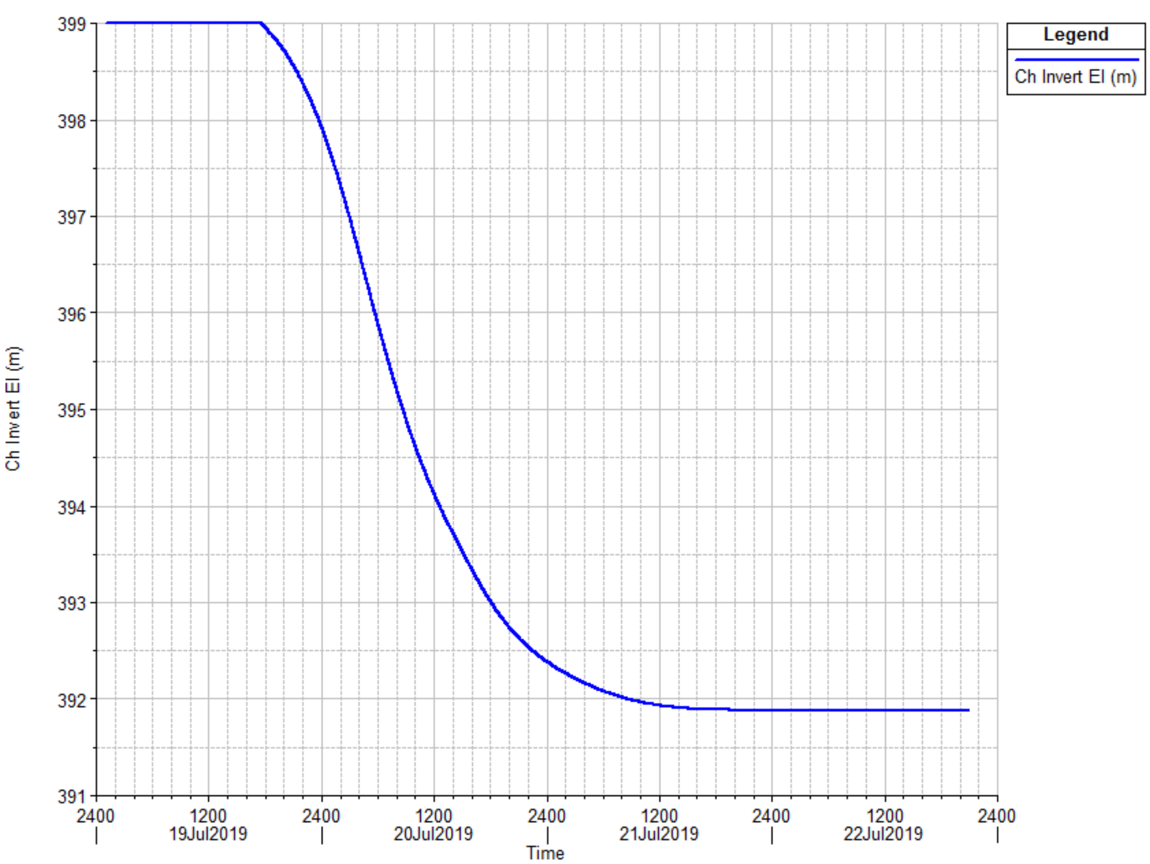


Figure 7-3: Time Series Showing Downcutting at Upstream End of Regime Channel

Figure 7-4 shows the hydrographs for immediately downstream of the regime channel breach. Peak discharge in the breach scenario is 385 m^3/s compared with 346 m^3/s for the non-breach scenario. The sediment transport and breach modelling demonstrates that complete failure of the regime channel armouring would result in an incremental rise in flood levels of approximately 0.25 m down to Forty Mile River compared to a flood rise of 1.4 m for the failure of CC1.

On this basis, it is concluded that CC2 reduces risk to public safety and the consequences of the failure of the structure are less than for CC1. It is also on this basis that Wood concludes the dam classification under CDA guidelines for CC2 should be “Low”.

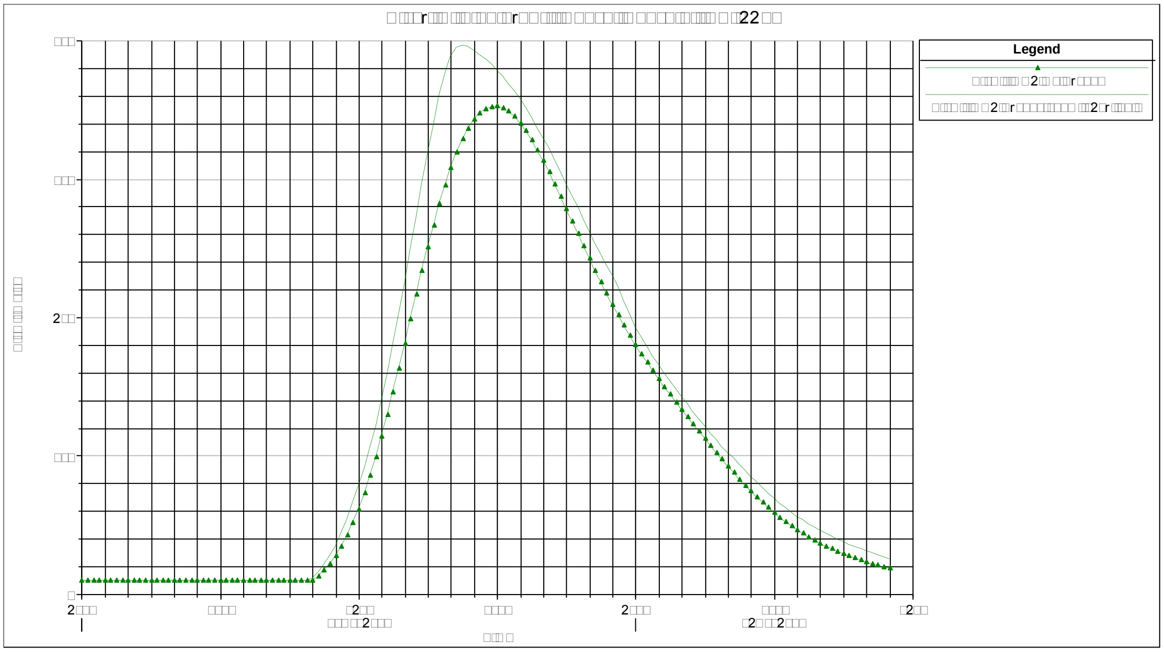


Figure 7-4: Comparison of Breach and no-Breach Hydrograph Downstream of Regime Channel for 1/3 between a 0.1% AEP Flood and the PMF

8.0 Summary and Closure

8.1 Summary of Findings

The findings of this report present modelling conducted on the flood-induced breaching of gabion drop structures on Clinton Creek and the modelling of potential erosion patterns for a proposed regime channel. The main findings of this report are as follows:

CC1 Spillway Breach Modelling

1. The severity of floods resulting from a breach of the drop structures depends more on the breach formation time than on the width or depth of the breach;
2. The resultant flood hydrograph is attenuated from approximately 1131 m^3/s immediately downstream from the dam to 560 m^3/s at the Yukon River;
3. Water levels rise in the incised reaches immediately downstream from the mine by up to 5 m; the impact on Forty Mile and Yukon River is an incremental rise in water levels of approximately 1.4 m, which is still well within bank levels of each river.

The conditions for classification as a "Significant" dam under CDA guidelines are confirmed by this study. Specifically:

- no permanent population is at risk;
- the habitat at risk is marginal and could be restored; and
- only recreational facilities, seasonal workplaces, and infrequently used transportation routes are at risk.

This study therefore concludes that appropriate design criteria for the CC1 Spillway IDF are as follows: 1/3 between the 0.1% AEP and PMF per the CDA 2014 guidelines.

CC2 Regime Channel Sediment Transport Modelling

The regime channel, if left unprotected by armouring, would erode by approximately 2.6 m in a 1% AEP flood, 3.3 m in a 0.1% AEP flood and 7.1 m in a 1/3 between 0.1% AEP and PMF. This material is shown in **Figures 7-2a** to **7-2c** to be deposited immediately downstream from the constructed regime channel (immediately upstream from Wolverine Creek). For the 1/3 between 0.1% AEP and PMF flood, the incremental rise in water levels is approximately 0.25 m.

In Wood's opinion, consequences of the failure of the CC2 regime channel are low since:

- No population is at risk and there is no possibility of loss of life other than through unforeseeable misadventure. The incremental increase in flood levels due to a breach of the regime channel would not change, in a significant way, overall risk to the population during an IDF event.
- There would be minimal incremental short-term and no long-term incremental losses to environmental and cultural value.
- The area is characterized by limited infrastructure and services and by low economic losses.

Wood concludes that the regime channel should be designed to a 0.1% AEP flood per CDA 2014 guidelines.

8.2 Report Limitations

The purpose of this assessment was to inform the 10% design phase for the six options under consideration for rehabilitating the Clinton Creek mine site. The final configuration of the selected options will have a profound effect on the results of any breach analysis. As discussed in Section 1.3, the work was undertaken primarily to support judgements applied in selecting preliminary design criteria for options.

Wood is confident that the analysis presented herein serves as a reasonable basis for judging these design criteria. However, the work was undertaken with data and information that may not be considered suitable for a comprehensive dam breach analysis. The analysis presented is therefore only suitable for use in the 10% design phase. Satellite-based topographic data used, including ArcticDTM data, present some gaps and uncertainties not suitable for the more accurate determinations of flood risk that may be needed to support final closure designs.

For the next design phase, Wood recommends that YG obtain accurate LiDAR-based DTM data for the whole of the Clinton Creek and Wolverine Creek valley to the confluence with the Yukon River. It is also recommended that a new PMF study and breach analysis be undertaken during option development to inform decisions on matters specific to those designs.

8.3 Closure

This report has been prepared for the exclusive use of the Government of Yukon for specific application to the area covered by this report. Any use that a third party makes of this report and any reliance on or decisions made based on it are the responsibility of such third parties. Wood accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report. The report has been prepared in accordance with generally accepted engineering practices. No other warranty, expressed or implied, is made.

Prepared for and on behalf of

Wood Environment & Infrastructure Solutions
a Division of Wood Canada Limited

Prepared by:

PROFESSIONAL
YUKON
GEOFF GRAHAM
TERRITORY
ENGINEER

13/11/19

Geoff Graham, P.Eng.
Associate Water Resources Engineer

Reviewed by:

Greg Standen, M.Eng., P.Eng.
Senior Water Resources Engineer

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INFRASTRUCTURE SOLUTIONS
SIGNATURE
Date 14 NOV 2019
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Engineers of Yukon

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Appendix A
Background to the Breach Assessment

The requirement for a dam breach assessment (DBA) came out of Wood's early considerations of candidate options in the summer/fall of 2018. At that time, it was posited that the status quo on Clinton Creek might be tolerable if the consequences of a waste rock dump breach fell within acceptable limits. Implicit in this consideration was the possibility of a revision, or addition, to the prescribed list of candidate options (i.e., adding an option that accepts the possibility of dump failure rather than mitigating that possibility as called for under CC1; essentially adding an option on the Clinton side that mirrored the largely status quo character of WC1 on the Wolverine side). Accordingly, the Dam Breach study was originally scoped (as Task 2 in TA13; December 1, 2018) as a study of the current configuration of the waste dump and its spillways; not of the design configurations that would apply to CC1 and CC2. Further, the dam breach study was originally scheduled to precede the development of 10% designs for CC1 and CC2, not to follow them.

During Project Workshop #2 (January 2019), Wood suggested that there might be benefits to revising and reducing the schedule of candidate options so that available resources could be focused on what appeared to be the most plausible alternatives. Following the workshop, the Partners considered this suggestion, and a supporting Discussion Paper prepared by Wood, and concluded that it was premature to discount any of the candidate options, notwithstanding doubts about the viability of some. This direction led to a reversion to the original scope for the balance of Wood's 10% design development effort. This focus on the six prescribed options effectively precluded the consideration of the variant that the Dam Breach study was originally predicated on. This re-focusing occurred after much of the basic hydrology undergirding the study had been completed or initiated.

The objective of the DBA evolved towards informing the engineering judgments that were required to apply hydrologic design criteria and decisions for the development of the Clinton spillway designs. The bulk of the hydrologic data and assessments that went into the DBA were applicable to this effort, if not directly, then as useful guidance for the application of engineering judgement to the 10% spillway designs. It was never intended that this version of the study would offer definitive representations of all design criteria for whatever option is ultimately selected on the Clinton side, but rather would inform those engineering judgments required to make the selection of a preferred concept.

The issues of resource and time constraints were also relevant to the conduct of the DBA and it is useful to highlight several issues, specifically:

- the DBA was not contemplated in setting the budget for the original 10% design phase MSA (May 2018), which meant that levels of design effort within the available funding envelope had to be adjusted to accommodate the study;
- the available funding did not provide for consideration of multiple breach scenarios for the Clinton design concepts (i.e., it provided for an assessment of the status quo, not multiple assessments of design concepts); and
- the original 10% design schedule did not accommodate breach assessments following the development of Clinton design concepts.

Finally, a more precise alignment between the DBA outcomes and the 10% design outcomes was not considered essential given the preliminary nature of this 10% design phase and its objectives (i.e., concept select, not final concept design/execution). It was understood that if more focused characterizations of breach consequences emerged as a requirement for the selected design concept, they could be incorporated into post 10% design development activity.

Appendix B

Comment Log

COMMENT LOG

Document Title: Clinton Creek Remediation Project - 2019 Clinton Creek Dam Breach Assessment

Document Date: 16 September 2019 **File No.** VE52705E

Comments By: J. Pigage (CIRNAC), A. Machica (AAM)

Responses By: B. Geddes (Wood)

Response Dates: 16 October 2019

Note 1 – Page numbers per commented document.

	Comment ID[1]	Comment	Response
1.	**j. pigage** **Section 3.0, Page 9** **10/04/2019**	CIRNAC is in agreement with the process followed for literature review, flood flow volume estimation (catchment and PMF determinations), and dam classification.	Noted.
2.	**j. pigage** **Section 6.3, Page 19** **10/04/2019**	Why does the dam breach methodology use the geometry of the existing temporary drop structures? Doesn't CC1 envision a spillway constructed using the IDF, not the existing conditions?	See Appendix A for explanation.
3.	**j. pigage** **Section 7.0, Page 29** **10/04/2019**	Why is there no breach assessment for the CC2 configuration? Hudgeon Lake remains at or around 400 m elevation, per CDA the landslide dam will remain a dam at this lake elevation. The findings of the breach assessment of current conditions (improper surrogate for CC1) indicate breach flows are more dependent on formation time than spillway geometry (Section 6.3.1). I would think a similar result is anticipated with a lower lake level, given the volume	A breach simulation using the downcutting rate of 0.23 m/hr was undertaken and further analysis added in Section 7.5. There was never an intention that the current condition model would be a surrogate for CC1 but rather that the analysis would inform the development of CC1. Wood has never concluded that any option would be infallible. The design intent for CC2 is to enable the stream to mimic as far as practical a natural regime channel and allowing the channel to change over time. Changes to the underlying permafrost may cause pools to form; gravels and silt could be washed in from the upstream catchment and the side slopes over time forming a more natural channel thalweg.

	Comment ID[1]	Comment	Response
		of water remaining in Hudgeon? On what basis does Wood conclude the regime channel is infallible?	
4.	**a. machica** **Section 7.0, Page 29** **10/07/2019**	The reviewer believes that justification for reducing dam consequence classification from "Significant" to "Low" has not been laid out sufficiently by Wood. The same downstream consequences exists whether it be 'Low' or 'Significant'. There is no clear argument that the 10 m permanent drawdown of lake level would result in: • no population at risk and no possibility of life loss other than unforeseeable misadventure • minimal short-term loss to environmental and cultural values • minimal economic losses	Further analysis has been added to the report in Section 7.5 to clarify Wood's position on this matter. The criteria for classification are based on incremental increase in flood risk due to a breach. Wood accepts that the first report issue did not have sufficient information to explain the rationale for reducing CC2 from a Significant to Low classification under CDA guidelines. We believe this has now been addressed.
5.	**a. machica** **Section 7.0, Page 29** **10/07/2019**	The reviewer is interested in knowing why there was no dam breach assessment conducted for both the proposed conveyance channel in CC1 and the regime channel in CC2. Their respective inundation zones could maybe provide sufficient comparison to justify the reduction in dam consequence from 'Significant' to 'Low'.	See Appendix A for commentary on the development of scope for the breach assessment.
6.	**j. pigage** **Section 8.1, Page 35** **10/04/2019**	Given a breach assessment was not conducted on the CC2 configuration it seems inappropriate to reduce the consequences of failure (dam classification) to "low".	This justification is now provided in Section 7.5.

	Comment ID[1]	Comment	Response
7.	**j. pigage** **Section 8.3, Page 36** **10/04/2019**	The findings of this Dam Breach Assessment are relied on to inform critical design components of each candidate closure option, specifically spillway geometries. The assessment of CC1 was completed on the existing temporary drop structures, not the CC1 spillway that is properly designed to the dam classification and corresponding flood event. There was no assessment of the CC2 configuration, yet the consequence of failure (dam classification) is reduced to "low", effectively changing the standard to which the CC2 spillway (regime channel) is designed. CIRNAC is concerned the above findings do not allow for the objective comparison of each closure concept. This amounts to a continuation of Wood's trend of presenting their preferred candidate closure option in a more favourable light than the others.	The reason for undertaking the CC1 breach assessment for the existing drop structures is explained in Appendix A. Wood has added Section 7.5 to explain the rationale behind concluding the dam classification for CC2 is considered Low by Wood. Wood does not have a preferred candidate closure option. Based on the preliminary breach assessment outcomes presented during the Project Partners meeting on 22 January 2019, Wood was of the view that a regime channel would likely reduce the flood risk in the event of a breach relative to a spillway. The analysis undertaken subsequent to that meeting provided the evidence that that supported that initial judgement. In the revised document, Wood has expanded the rationales grounding judgements relating to consequence classifications. Wood acknowledges that these are still judgements that will require degrees of validation during the future design development efforts that will follow concept select.